ENGINEERING RECORD

A WEEKLY JOURNAL DEVOTED TO CIVIL ENGINEERING AND CONTRACTING

PUBLISHED BY THE McGRAW PUBLISHING COMPANY, INC.

Volume 68 NEW YORK, SATURDAY, NOVEMBER 1, 1913 Number 18

Ventilation of Small Theaters The length to which opponents of adequate ventilation of small theaters are going in Chicago to nullify the efforts of Dr. George B. Young, the health commissioner, in enforcing a really commendable ordinance indicates the need of educating along this line those architects and builders who consider their work done when four walls have been thrown together. Many "nickel" theaters erected after this formula are so constructed that ventilation cannot be provided without elaborate equipment or reconstruction. Consequently the owners are protesting most vigorously against Dr. Young's crusade and are working to repeal the ordinance. Moving-picture theaters which receive the approval of the health department are allowed to display a sign to that effect. The commissioner's hands should be upheld and precautions should be taken to prevent the construction of buildings with inadequate provision for ventilation. Prevention is always better than cure.

The Transcona Elevator The sinking and tilting without destruction of the 1,000,000-bushel concrete grain elevator of the Canadian Pacific Railroad at Transcona, a suburb of Winnipeg, is fortunately a most unusual occurrence. It recalls a similar accident, the tilting of a warehouse in Tunis some years ago, which was described in the Engineering Record of Nov. 3, 1906. The elevator at Transcona consisted of sixty-five circular and forty-eight interstice bins and was erected last fall. On Oct. 18 movement was first noted. It continued for 12 hours, the elevator sinking 20 ft. on one side and reaching an inclination between 30 and 35 deg. with the vertical. Then the cupola, a steel structure, collapsed, and the removal of part of the load checked the movement. The structure is apparently but little damaged, so far as the concrete construction is concerned. The bins were filled at the time. No authoritative data as to the soil conditions or tests previous to the construction of the elevator have been given out, but it is commonly reported that the unreliable character of the soil was indicated recently by some trouble in a subway not far from the elevator. Authentic and complete reports on the accident will be awaited with interest by all engineers.

Juggling with Riparian Rights Riparian property owners along watercourses where Federal river improvements are in progress should keep a watchful eye on the changes the engineers make in the position of the navigable channel. River improvements, being mostly under water, attract little attention, but a decision by Judge Morris of the U. S. District Court (188 Fed. Rep. 356) indicates that these improvements may occasionally have an effect on property which is startling in its novelty and worth noting as one of the by-products of engineering. An owner of riparian lands on a navigable stream has an absolute title down to the water's edge. The land under water, to the middle thread of the main natural channel, is also owned in a sense by the riparian property holder, subject to the paramount right of the State in the stream for the purpose of maintaining and improving its navigability. These two rules of law are well known, but Judge Morris has decided that they are subject to the following important condition: Where the Government, in the exercise of its right to improve the navigation of a river, changed the channel to the opposite side of an island rising from the river bed, this change transferred the title to the island to the opposite riparian proprietor. It is hardly necessary to say that an appeal to the U. S. Circuit Court was taken from this ruling, which so jauntily adds the ponderous dredge to the natural disturbers of the property rights along our navigable streams.

Groined-Arch Construction A groined-arch failure like that in Baltimore last week is to be expected occasionally, for hardly anybody who has to do with the actual construction of such roofs in the field appreciates how very important it is to pay minute attention to every detail of the work. A groined arch is not susceptible of satisfactory mathematical analysis, and its design is usually guided by the experience of the engineer plus whatever hints he has been able to pick up from an observation of the designs worked out by others. Only a few years, relatively, have passed since this type of roof came into use in this country, at least in waterworks, and the failures which have occurred show that the arches must not only be built with exceptional care, but also supported for a considerable period, if danger of collapse is to be avoided. The uncertainty regarding the distribution of stresses through a groined roof is unfortunately equalled by the certainty that such a roof, unless given unusual dimensions, is not adapted to resist horizontal stresses and vertical loads which are not uniformly distributed. Out of these conditions have come the two well-known construction rules: Keep the centers in place under the last five rows of arches poured, at least; and, be exceptionally careful in placing the earth cover on the roof to avoid any local overloading. A third rule might well be added: The quality of the concrete placed in such roofs must be maintained at a uniformly high excellence. A concrete structure of such indeterminate analytical properties must be surrounded with every safeguard.

Small Details in Paving Work Observant engineers need not be told that in small details lies the success of pavement construction. Except to the experienced these seem of little importance. The two refinements noted below are simple of statement, but on them depends to a measure difficult to appraise the success of pavements subjected to exceptionally severe traffic, and guaranteed for five years. Both were noted on New York paving jobs. One of them was in the laying of 3 in. of asphaltic concrete on Riverside Drive. The departure from usual methods consisted in rolling the hot mixed top in two layers of 1½ in. each. This, of course, is more expensive for the contractor than one rolling, but is more than justified by the results in the case of pavements built to support the heaviest traffic. The first or lower layer was rolled sufficiently to smooth it out, but ultimate compaction was not attempted. On this course before it had cooled was placed the top layer, which was rolled until, examination has shown, there was secured a degree of density that would have been impossible if the entire 3-in. top had been rolled at one time. The heat of the top layer combined with that of the lower secured so perfect an amalgamation that no dividing line is observable in cross-section samples. It is important that not more than, say, 10 yd. of the lower layer be put down before the top is ready to spread and roll. A paving "kink" of quite a different sort developed in a close examination of the sand gradings for a sheet-asphalt pavement where the motor traffic is particularly heavy. The pavement had been "hardened" by an unusual percentage of Portland cement filler in place of ordinary dust. The object was to provide against the softening effect of motor-car grease, oil drippings and the heat of automobile tires, which, with the passing of several thousand cars a day, might be destructive if no antidote were provided. Both of these variations from standard practice and specifications emphasize the importance of the highest degree of care and skill in paving work where the service is severe.

"Big Business" and Regulation Discussing "big business" in its relation to industrial prosperity before the American Mining Congress in Philadelphia last week Dr. Charles R. Van Hise, president of the University of Wisconsin, laid emphasis on two points in the trust problem which, while not new, deserve occasional emphasis, for one of them is an index to the inexorable laws of large industrial development, while the other points to a frame of mind which must be changed before there will be fair treatment of the whole problem. Referring to the mining industry Dr. Van Hise pointed out that the competition enforced by the Sherman law has caused losses on zinc and lead in Missouri and Wisconsin mines as high as 45 to 50 per cent, while about 50 per cent of anthracite and bituminous coal is being left in the ground by the methods forced upon the operators by the competitive system. In other words, a law framed fundamentally to conserve the interests of the people is, by enforcing destructive competitive methods, nullifying this conservation, such as it is, by irreparable economic losses in other directions. As the New York "Times" said in discussing the address, "it is being learned at last that business makes laws which legislatures cannot displace." The logic of Dr. Van Hise's remarks points plainly to regulation of monopoly in industry, just as to-day there is control of monopoly of public-service corporations. The other point in Dr. Van Hise's address to which reference is made needs no comment, especially after the attention given the subject in these columns last spring. It is pertinent to note, however, that it indicates the attitude of those upon whom we rely to do justice to the whole people. Referring to the rider in the sundry civil bill stipulating that no part of a $300,000 appropriation for enforcing the anti-trust laws should be used in prosecution of labor and farmers' unions, he says: "I am willing to stand for enforcement of law when the law is enforced alike for all; but when somebody is picked out because he is in the front seat, or because it is good politics to attack him and allow 99 or 999 others to escape, I say that it is a profoundly immoral situation, and that is exactly the situation in this country."

Use of Forest Lands

Reference is made elsewhere in these columns to an important constitutional amendment to come before the people of New York State next Tuesday. One other amendment to be voted on on the same day has received less attention, but deserves more than passing consideration. It would permit the use of 3 per cent of the State's forest reserves for the construction of reservoirs, the water to be used for municipal supplies and the State's canals. The use of State lands for such purposes is entirely commendable, but it is not amiss to indicate one of the dangers involved because of the present frame of mind of New York politicians. As may be remembered, strenuous efforts have been made to put a law on the books permitting the construction and operation by the State of hydroelectric plants on the Barge Canal. The faction back of this movement will welcome the passage of the amendment, for it would mean more water for its hydroelectric schemes, the viciousness of which was plainly pointed out in this journal last winter. Furthermore, some of the advocates of the proposed amendment have not hesitated to admit that such use might well be made of the surplus waters. While, therefore, the passage of the amendment is to be desired, the uses to which the water will be put need to be watched carefully. The amendment, moreover, calls attention to another item in the New York Constitution that might be amended with profit. The fundamental document now declares that the timber in the reserves shall not be sold, removed or destroyed and that the lands "shall forever be kept as wild forest lands." Here is good work for the conservationists. To condemn forests to disuse forever is not in harmony with progressive ideas as to the economic use of natural resources, and sooner or later this archaic clause must yield to a provision for the working of the State's forests on a scientific forestry basis.

Wisdom in Numbers

Gratuitous engineering advice, paid for by unofficial bodies, too often meets with scant courtesy from municipal officials. In Chicago last week a report was rendered to a council committee on railway terminals by Mr. John F. Wallace. He was asked to take up only the matter of one terminal, that of the Union Station Company. A group of citizens believed that the whole terminal situation should be studied, so that when other terminals were replaced this station would fit in properly with the transportation system of Chicago as a whole.

Mr. Bion J. Arnold was chosen to make this study. Although Mr. Wallace went into the general matter to a certain extent and proposed two foci for further development of passenger stations, it would seem right and proper for the council committee to await Mr. Arnold's report, which he states will be ready within a month.

It is not unlikely, as this journal has said on a previous occasion, that Mr. Arnold will approve in a general way the report of Mr. Wallace, but, as the latter approved the proposals of the railroads with some very commendable changes, Mr. Arnold may make further beneficial modifications in the light of more extended study of the general situation. For instance, Mr. Wallace touches very lightly on the interconnecting railroad between the two great stations he would build at diagonal corners of a one-mile square. He only hinted that at some future date suburban trains might be looped around the business district. He states that only 2 per cent of the through-passenger traffic would need to change stations, but no figures are given of the increasing suburban traffic which would welcome any method that might bring it closer to its final destination. That a railroad loop, unless depressed, is the solution will be questioned by those who are now clamoring for the removal of the present elevated structure.

Rushing the ordinances of approval through before the more comprehensive report is available savors too much of those steam-roller methods made memorable in Chicago in the not-distant past.

Developing Engineering Executives

In the successful management of the modern large industrial corporation the progressive development of the subordinate officials to meet the larger responsibilities of the executive leader is a problem which is never long put aside. The value of personality in business was never greater than it is to-day, but the complex demands of science and industry cannot be satisfied, in the long run, by depending upon the individual alone. Organized effort is the only means of accomplishment in the larger industrial advance of the times, and the encouragement and training of subordinates are vital to enduring success. The development of engineering executives is a phase of this matter which deserves a more general consideration.

Too often in the past the duties of the engineer have been purely advisory in industrial affairs. He has been called in to work out the details of plans roughed out in advance by the "business" side of the establishment and often has had to sponsor reports and designs which never would have been perpetrated had he been accorded a place at the council table where the project was first discussed. In the worst cases that occur a non-technical board of directors may give the engineer no opportunity whatever to display his initiative and specialized judgment, but may commit the organization to an ill-advised program through inadequate consideration of estimates, too great haste to get contracts signed, or inordinate zeal to practice the pet hobby of this or that influential member.

Plenty of industrial corporations have long since broken away from these policies and have learned the wisdom of developing the engineering executive with the ability to deal in a large way with matters of technical importance. They have gone about this work in a liberal fashion by removing many of the burdens of detail from the shoulders of the chief of the engineering department; by authorizing the selection and training by him of understudies of promise; by minimizing interruptions of his most responsible work for the sake of minor results, and by sharing with him the proceedings of the conference room. They have encouraged him to travel extensively in connection with the study of other plants and have backed him in every reasonable and proper effort to increase his efficiency as a high-quality employee. In broadening the engineer's duties these organizations have not been parsimonious in the supply of first-class clerical assistance and the installation of labor and time-saving office systems, but, most of all, they have given him a real share in the conduct of corporate affairs and have developed his ofttimes latent executive powers by remodeling the form or redistributing the duties of the engineering department and its subdivisions. Some of the results of such policies have been remarkable in the direction of increased net earnings growing out of new schemes for the development of business in fields unappreciated by minds without technical training, and arising, also, from betterments the possibilities of which were not apparent when the eyes and hands of the engineer were too closely limited to the consideration of routine and intra-departmental problems.

A word to the engineer himself is desirable as he crosses over into the region of executive interests. There is some danger in dealing constantly with large schemes and plans en masse of losing the power to appreciate detail, and, on the other hand, there is a chance that insufficient reliance upon the work of subordinates will make the new duties seem heavy beyond endurance. Human limitations must be recognized and a happy medium sought in handling both details and men. Significant details must be studied with as much care as before executive tasks came so much to the front, and the observation of subordinate abilities must become intensified. Ability to detect essentials must be exercised as never before, and every feasible device for saving the executive quarter-hours must be tried out. By working along such lines and gradually depending more and more upon responsible assistants for the execution of plans and investigations, without losing sight of the vital points, the engineer-executive comes into his own sphere of maximum usefulness.

Excess Condemnation—An Important Municipal Issue

The desirability of permitting "excess condemnation" of property for public improvements, which must be passed upon by the voters of New York State at the approaching election, is something not generally understood, even by many who usually join the ranks of those striving to better the administration of public affairs. The present method of paying for condemned real estate in New York is based on Article 1, Section 6, of the State Constitution, which says that the damages must be "just compensation." The courts have repeatedly stated that this is to be measured by the difference in the market value of the premises before and after the appropriation, where only a part of the property is taken, except where benefits are involved, when owners must be awarded at least the market value of the land taken. "This rule is easy of expression but difficult of application," the New York Court of Claims remarked feelingly in an opinion rendered two years ago, and every city official who has to carry on condemnation proceedings will agree with this assertion. The courts have struggled valiantly to bring about some sort of uniformity of procedure for assessing the damages, yet the conditions of the different cases vary so widely that little has been accomplished. For example, it has been laid down repeatedly in legal treatises and judicial decisions that, to quote from one decision, "the profits of a business are too uncertain, and depend upon too many contingencies, to safely be accepted as any evidence of the usable value of the property upon which the business is carried on." And yet in a great many cases in the New York courts it becomes absolutely necessary to pay some attention, and occasionally a great deal of attention, to the profits of the business, for they are manifestly affected by its location.

The difficulty of making a fair appraisal of the value of condemned property is materially enhanced when only a part of the property is taken. The reduction in area of the property in this case works all kinds of injury to the owners' remnants, and they deny with indignation the possibility of any benefit ever accruing to their land from the public improvement, which is the cause of all the trouble. In fact, the conditions are such that a city which condemns parts of properties must usually pay exorbitant prices for what it gets, must occasionally leave little useless strips and triangles along its right-of-way, and must witness with chagrin the sale of the larger remnants at prices in excess of those paid for the entire properties not long before the improvements were inaugurated. In some cases the opportunities for private gain are so great that improvements like the construction of bridge approaches have been held up for many months until the property along the routes of the approaches was acquired by friends of the administration. In addition to this temptation to grafters the constitutional inability of a city in most States to condemn more land than an improvement actually requires makes it impracticable to open thoroughfares which will benefit both congested districts that are cut and through routes which are linked together, to straighten crooked streets, and to make usable the interior of blocks which are too large for proper development without cross-streets.

No city should engage in real-estate speculation, it is true, but no city should be prevented from developing its streets in the most economical and efficient manner. Experience in Europe has demonstrated convincingly that the best way to proceed in such improvements is for the city to take the land needed for the betterments and also enough adjoining property to make it certain that the plots of land abutting on the street are in usable shape. The revenue to the city from the sale of these improved plots has occasionally proved of great help in meeting the costs of the improvements. Five American States—Massachusetts, Pennsylvania, Ohio, Michigan and California—already permit this excess condemnation. The great need of such permission in connection with the public improvements of the City of New York has led the municipal authorities to request the voters of the State to amend the constitution to the extent of permitting a city to take, if it sees fit, more land than is needed for actual street improvements, provided the additional property shall not exceed what is required to yield good building sites. The great merits of this desired legislation are opposed by only a single defect, the possibility that grafters in city offices might utilize the power of excess condemnation to enrich themselves and their friends by forcing the city into what would amount to extensive real-estate operations. As a matter of fact, the opportunities for such gain will probably not be greater under the new law than under the old, and will apparently be considerably less on account of the publicity which must attend all the condemnation proceedings under any workable plan of utilizing the right of excess condemnation.

This is the second time the voters of the State have been asked to give this right to city officials, and now that the facts of the matter are better known there seems to be a fair show that New York will fall into line at this month's election with the five States which have already put themselves on the side of businesslike administration. Even if the vote is again adverse, the principle of excess condemnation is so important that it will doubtless be preached more often and more widely than heretofore and will be a more prominent issue at the election two years hence. It is one of those procedures which must inevitably become legalized, and it is desirable for the rational progress of certain classes of municipal improvements that it be legalized now.

Power Economies in Large Generating Plants

The well-known advantages of concentrating generating facilities in the business of electric-power supply may appear a trite subject for discussion along general lines, but as a matter of fact the last word on this topic will remain to be said for a good while to come. The problem of securing the cheapest possible power service is always with us, and in the investigation of each specific case there are in many instances useful opportunities to define the limits of practice beyond which it does not pay to go in selecting equipment or in considering offers from organizations making a specialty of power supply. In a paper on the "Relation of Plant Size to Power Cost," presented by Mr. Paul M. Lincoln at a recent meeting of the American Institute of Electrical Engineers, some suggestive sidelights appear on the limitations of small generating stations on the score of economy of operation, and the author's deductions, viewing the problem from the standpoint of unit analysis, are decidedly significant, despite the fact that the paper contains few quantitative cost data.

Broadly speaking, there are three chief reasons why a large plant can take care of a given service more economically than a small one: (1) Because the initial cost per kilowatt of capacity in large units is far less than in a small installation; (2) because a large plant inherently can be operated more economically than a small one, affording economies through the use of apparatus of prohibitive cost in a little plant; (3) because of the existence of the diversity factor, whereby 1 kw of capacity in a central station will serve a combined load requiring considerably more than this capacity where each part of the combined load is supplied separately. The author presented a number of logarithmic curves showing the relation between first cost and the capacity of generating apparatus, transformers and boilers, the saving in unit investment being very marked with modern equipment as the size increases. Although this relation is usually recognized qualitatively, few engineers have an adequate conception of how much it means quantitatively. Thus, for the same speed, the cost per kilowatt in a generator will decrease by approximately 65 per cent for a tenfold increase in the size of the unit; a fivefold increase in the capacity of a 22,000-volt, 60-cycle transformer results in a 53 per cent decrease in its unit cost; and a tenfold increase in the size of a transformer for 110,000-volt service cuts the cost per kilowatt 76 per cent. A 40 per cent decrease occurs in unit boiler cost where the capacity is increased tenfold. This sort of analysis is highly suggestive to the power engineer, and while such figures cannot be used without some discrimination, they are worth collecting by people for whom power costs have some significance, provided the limitations of unit analyses are plainly recognized. It is obvious that no curve showing the relation between unit cost and the size of apparatus can be pushed too far in practical work. For example, if one attempted to extend the author's curve for 110,000-volt transformers to obtain the cost of a 1-kw unit installation, the result would manifestly be absurd.

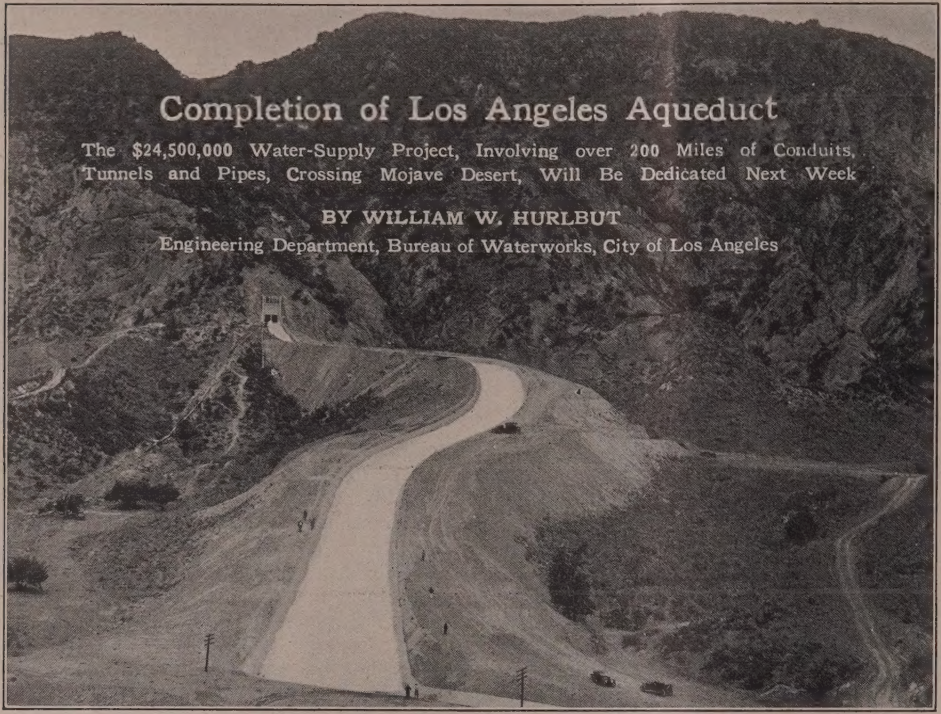
Economies in fuel utilization are almost always available in a large station to an extent difficult, if not impossible, of realization in many small plants. The inherent fuel economy of condensing units as compared with non-condensing apparatus is nearly in the ratio of 2 to 1, yet the limitations of water supply often keep the small plant from enjoying the former. As plants increase in size a point is soon reached where the investment required to secure the economies of auxiliary apparatus in the way of superheaters, economizers, mechanical stokers, automatic or semi-automatic coal and ash-handling machinery, special forms of grates and furnaces for burning uniform fuel and other incidental but important resources of the modern designer of efficient stations is justified. To determine the dividing line where it pays to install such equipment is a problem whose solution rests in no small degree upon the study of unit costs and operating results in a variety of cases, and it is in analyzing individual detailed power-cost situations that the engineer gains experience and judgment that enable him to deal intelligently with such questions. Many of these devices for increasing operating economy do so by saving labor, and the author's point that their introduction is an engineering mistake in stations of such small size that a minimum fixed attendance is established regardless of their use deserves careful consideration in figuring power costs.

The benefits of the diversity factor in the large plant and the value of being able to operate generating units closer to their ratings than in many smaller installations need not be elaborated here, other than to refer to the use of hydroelectric energy for house heating in Idaho during the winter and for irrigation in summer as a striking example of the value of this factor. The possibilities of handling railroad loads in lighting and general power plants are well known, even if this service is largely in its infancy. Mr. Lincoln's contention that the cost of distribution is seldom anywhere nearly enough to offset the economies, including all fixed charges, of centralized power production is well established to-day. Not so well realized are his remarks anent the economies of central-station service even in many plants where exhaust-steam heating and local electrical service are closely related. The fact is that in many of these installations there is a local diversity factor which actually works against the economical operation of the local plant, arising from the fact that the greatest demands for heating and lighting service, as in an office building, are non-coincident, to say nothing of the waste of exhaust steam during the summer season and for many weeks before and after it. It seems pretty well demonstrated that the central station can supply service to most small consumers and to many large ones more economically than these can furnish themselves with electricity, and while no fixed rule can be given for the choice of power supply for the former, the study of each case with the relations outlined in Mr. Lincoln's paper in mind is important in the wise selection of service.

Completion of Los Angeles Aqueduct

The $24,500,000 Water-Supply Project, Involving over 200 Miles of Conduits, Tunnels and Pipes, Crossing Mojave Desert, Will Be Dedicated Next Week

BY WILLIAM W. HURLBUT
Engineering Department, Bureau of Waterworks, City of Los Angeles



THE Los Angeles Aqueduct, one of the most extensive developments ever undertaken for a municipal water supply, and from an engineering standpoint one of the boldest in existence, considering its great length, the topography and the extreme climatic conditions encountered and overcome, will be formally dedicated on Nov. 5 and 6. The total funds voted for this work amounted to $24,500,000, and with the exception of 9 miles of construction in the South Antelope division, two short steel siphons in the Jawbone division and the commissary and medical departments, which were under contract, the entire project was built under the direct supervision of the city. The works have been completed within the bond issue and estimated time, and the plans, as outlined by the consulting board of engineers, have been greatly enlarged upon. The Los Angeles Aqueduct stands as a monument to municipal construction.

In addition to the waterworks system for municipal purposes, the surplus waters will provide for the irrigation of 135,000 acres of land near the city and for the future development of 120,000 hp, maximum, of electrical energy.

Financing Project

In the latter part of 1904 the remarkable increase in population and consequent increased water consumption made it necessary for Los Angeles to supplement its supply from the Los Angeles River. The Owens River project or the Los Angeles Aqueduct was investigated during 1905. In September, 1905, bonds to the amount of $1,500,000 were voted to buy up options on land and water rights and carry on the preliminary engineering investigations. During the fall of 1906 the plans and estimates were completed under the supervision of Mr. William Mulholland, then superintendent of the Water Department, assisted by Mr. J. B. Lippincott. A consulting board of engineers was appointed, composed of Messrs. Frederic P. Sterns, John R. Freeman and James D. Schuyler. After several months of investigation this board approved the project outlined by Mr. Mulholland.

In June, 1907, bonds to the amount of $23,000,000—the amount estimated by Mr. Mulholland to complete the work—were voted by the people. Table 1 gives the classification of work as constructed and completed.

Table 1—Classification of Aqueduct Work

Classification	Miles
Unlined canal	23.726
Lined canal	37.054
Covered conduit	97.641
Tunnels	42.903
Siphons	12.052
Flumes	0.165
Haiwee By-Pass	2.001
Reservoir	7.870
Power tunnels and waterway	9.845
Total	233.257

The power construction noted in Table 1 is a joint undertaking with a pro rata of cost paid from the power-bonds issue of $3,500,000. There are 9.23 miles of tunnels in this power development through the mountains bordering on San Francisquito canyon. Of this portion of the work there still remains less than ¼ mile of tunnel to excavate and 2 miles of concrete lining to be placed. However, this does not interfere with the operation of the aqueduct as a domestic supply.

The intake of the Los Angeles Aqueduct is located in the heart of Owens Valley, about 12 miles north of Independence, as shown in the map. This valley is 120 miles in length and from 6 to 12 miles in width, and the elevation varies from 6500 to 3500 ft. The area of the basin is about 2800 sq. miles. The valley receives the discharge from the Sierras for a distance of more than 120 miles from some forty lateral streams. The normal annual rainfall in the valley is only 5 in. and the increase with the elevation to the crest of the Sierras is in excess of 40 in.

In the Owens Valley the city has acquired through purchase and Government filings 124,929 acres of land and a grand total of 135,116 acres for the entire project, exclusive of rights-of-way for the aqueduct, at a total cost of $1,460,734.

The Long Valley reservoir site is located on the headwaters of the Owens River and tributaries, about 50 miles north of the intake. The purpose of this reservoir is to control the flow of the Owens River and tributaries in years of excessive discharge. The elevation of the maximum water surface is 6810 ft., with a dam 160 ft. high and 520 ft. in length. It has a capacity of 340,980 acre-ft. and a surface area of 8686 acres. The reservoir would furnish a continuous flow of 430 sec.-ft., the capacity of the aqueduct, for a period of 427 days.

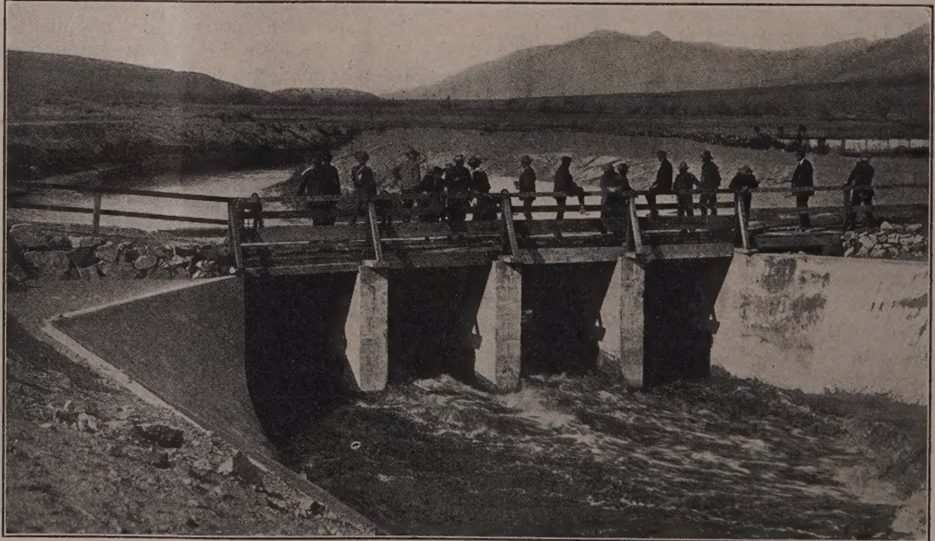
The Tinemaha reservoir site, located 6 miles north of the intake, acts in the same manner as the Long Valley site. The maximum water surface is 3887 ft., the capacity 127,325 acre-ft., the surface area 7074 acres, and the dam 40 ft. in height.

If it is found necessary to construct for future needs, the city controls, in addition to these reservoir sites, an artesian well belt about 40 miles in extent. There have been twelve wells sunk from 200 to 400 ft. in depth which flow from 1 to 2 sec.-ft. per well.

The controlling elevation at Black Rock Springs, with a flow of 23 sec.-ft., fixes the elevation of the intake at 3812 ft., in order to divert the flow of this spring into the canal. The Haiwee reservoir, with a maximum water surface at El. 3760, controls the grade from the intake to Haiwee, a distance of 60.78 miles.

Intake and Diversion Canal

The unlined canal extends for 23.726 miles through the moist bottom lands of the valley. It has an average depth of 10 ft., a bottom width of 38 ft., a top width of 62 ft., and a capacity of 801 sec.-ft. It was excavated with two floating suction dredges and one dipper



Opening Gates at Aqueduct Intake to Admit Water to Canal

dredge, all electrically operated. About 60 per cent of the trench was excavated with the two suction dredges at a cost of $3.10 per lineal foot, direct field charges, while the charge with the dipper dredge was but 80 cents per lineal foot.

The average excavation per lineal foot for this diversion was 15 cu. yd. with a cost for all operating charges of $1.80 per foot, and a total cost, including administrative and auxiliary charges, of $3.14 per foot. The hydraulic gradient on this section is 1 ft. in a mile, and extends to the north end of the Alabama Hills. The seepage gain in the canal is about 8 sec.-ft., exclusive of the Black Rock Springs flow, between the intake and the head of the Alabamas.

The headworks consist of a diversion dam and weir, 325 ft. in length, with two sets of three radial gates at each end, extending to the bottom of the river channel, the entire weir being built of 24-ft. interlocking 12-in. sheet piling and concrete. At the intake proper are four radial gates, 7½ x 8 ft. The feature of the headworks is the simplicity of construction, the total cost amounting only to $63,350.

As to the quality of the water, the average dissolved salt content of the Owens Valley water is from 15 to 17 grains per gallon, which is about one-half the mineral content of the present water supply, 31 grains per gallon.

Table 2 shows analyses of Black Rock Springs and Owens River water by Prof. Laird J. Stabler of the University of Southern California, taken April 19, 1912.

TABLE 2—ANALYSES OF BLACK ROCK SPRINGS AND OWENS RIVER WATER, GRAINS PER GALLON

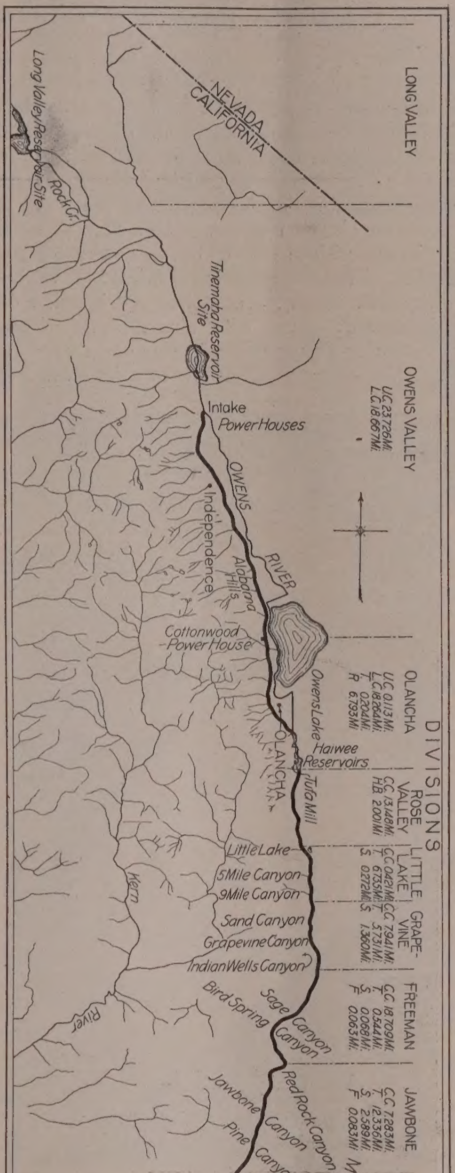
	Black Rock Springs	Owens River at intake
Sodium chloride	0.92	3.05
Sodium sulphate	1.97	4.47
Sodium carbonate	1.64	4.57
Calcium carbonate	3.42	7.71
Magnesium carbonate	0.64	1.26
Iron and alumina	0.23	0.36
Silica	0.16	0.54
Total dissolved salts	8.98	22.96

Analyses by the U. S. Geological Survey of samples from Round Valley and the intake, taken daily for the year 1908, show 19.14 grains per gallon, according to Water Supply Paper 237.

ELECTRIC POWER SHOVELS

At the head of the Alabamas the ground drops very rapidly, and for a distance of about 16 miles the aqueduct follows well up on the face of the hills, with great detours around the extensive deposits of the alluvial fans. The excavation in the Alabama Hills section was done with both power and steam shovels. The material excavated on the aqueduct was loose desert soil, soil and boulders from 3 to 5 ft. in diameter, large boulders and cemented gravels, and conglomerate and solid rock. It was necessary to shoot the rock and very often boulders as large as 30 ft. in diameter were encountered.

The electric shovels operated most economically in the desert soil. Their cost was about 25 per cent of the cost of fuel oil for the steamers. This class of work averaged about 8 to 10 cents per cubic yard for the power and steam shovels. The small boulders could not be handled by the power shovels without shooting in advance; this operation cost about 12 cents per yard. The steamers were able to handle this class of work on an average of 18 cents per yard. Boulders weighing from 6 to 10 tons were pushed from the ditch by the steamers without any difficulty.



Map Showing Route of Los Angeles Aqueduct (Heavy Black Line) and Classification of Work on Divisions

In this division deep, side-hill cuts were encountered. At some places the top cuts were as much as 70 ft. In order to handle this class of work two shovels in tandem were operated to advantage, a model 40 Marion shovel cutting down the upper half and throwing the material down the hillside, while a model 60 Marion shovel followed in and excavated the trench and would pick this material up and throw it down the hill, completing the excavation.

The excavation of this section was of the most difficult encountered on the entire work. The lined canal from the head of the Alabamas to the Haiwee reservoir is 37.054 miles long. Concrete was placed immediately behind the shovels on a 1 to 1 slope in 6-ft. sections without forms. The bottom was floated in and formed with templets. The sides were plastered. The temperatures in this region are extreme. During the winter it was necessary to take the chill off the water to mix and hasten the setting of the concrete, which was then covered with 2-in. planking, helping the concrete to retain the heat and preventing the frost from affecting it.

The costs for this work per lineal foot follow, and are remarkable, taking into consideration the difficulties encountered.

TABLE 3—COST OF LINED CANAL

	Direct field charges and auxiliary construction	Total, incl. gen'l, administrative, land, etc.
Alabama Hills	$10.858	$12.43
Olancha Division	9.512	10.87

The capacity of the canal from the Alabamas south to Haiwee is 900 sec.-ft., the additional 100 sec.-ft. capacity being allowed to divert the flow of the side streams.

At Cottonwood Creek an arched overhead waterway crossing is provided to take care of flood water of Cottonwood Creek, and there are weirs to divert the flow into the aqueduct. Here also is located a standard wasteway so that the entire flow of the aqueduct can be diverted into the channel of Cottonwood Creek. A similar structure, at the head of the Alabamas, serves both as a sand-box clean-out and wasteway. At the north end of Haiwee reservoir is an overflow weir to regulate, if necessary, the flow before entering the reservoir. The base of the weir is at the maximum water surface level of the reservoir.

The general practice was to locate so that the top of the finished conduit was even with or below the original surface of the ground, while on side-hill work the upper wall is below the original surface and the lower wall in from 3 to 5 ft. of back-fill.

There is located on the aqueduct less than ¼ mile of fills. Where fills were necessary, sufficiently large culverts were built to care for the drainage and fills made were then flooded from one to two years before the conduit was built across them. The general policy was to locate so as to pass storm water overhead rather than underneath with expensive culvert construction.

On Feb. 13 of this year the diversion gates at the intake were opened and the water allowed to proceed to the Haiwee reservoir. A continuous flow has been pouring into Haiwee since that date, and at present there are stored there 40,000 acre-ft. The first 61 miles of the aqueduct, therefore, have been in operation for eight months.

HAIWEE RESERVOIR

The Haiwee reservoir is located at the extreme south end of the Owens Valley, in what was the ancient channel of the Owens River.

It is 7 miles long and from ½ to 1 mile in width, with a surface area of 2100 acres and a capacity of 63,800 acre-ft. In the center of the reservoir site is a summit and the topography slopes to the north and south from this point. Consequently there are two dams, the north and south. The north dam has a center height of 32 ft., side slopes on water face of 3 to 1, and on up-stream face of 2½ to 1. The length along the crest is 1890 ft. The total material placed was 168,372 cu. yd.; 26,000 cu. yd. were placed by teams and the remainder by hydraulic fill. The material placed with teams cost 30 cents per cubic yard, while the hydraulic material cost but 8½ cents per cubic yard.

The water face of the dam is paved with 8 in. of concrete laid in slabs 5 x 10 ft. and separated with asphaltum expansion joints. The total cost of the dam was $49,700, of which $17,500 was for paving the water face with 3200 cu. yd. of concrete at a unit cost of $5.40 per yard.

On the south dam it was necessary to carry the core-wall trench to bed-rock, a depth of 120 ft. below the ground surface. The first 14 ft. in depth was excavated with a power shovel and the remainder by hand; 27,000 cu. yd. of material were excavated from the trench at a cost of $1.04 per yard. It was necessary to timber the entire trench at a cost of $41,400, or $1.54 per cubic yard of trench excavation. The core-wall is a clay puddle and the material was hauled with teams, the average haul being about ¼ mile at a cost of 32 cents per yard.

HYDRAULIC SLUICING

A sufficient head of water was not obtainable for hydraulic sluicing, and it was necessary to handle the material by railroad. Three trains of seven cars each, double-dump type, were operated, the capacity being 28 yd. to the train. The tracks were carried along both sides of the dam where the material was dumped. The earth then was washed down with a 2-in. nozzle operated from a steel float equipped with a 6-in. motor-driven centrifugal pump. The fine clay and silt were washed and deposited in the center of the puddle, the coarser material remaining along the outside slopes. Two eight-hour shifts were used and handled from 400 to 500 cars per day. The cars were loaded with a Marion model 60 power shovel, equipped with a 2½-yd. bucket making about three swings to the minute. The material composing the fill is a decomposed shale, about 30 per cent clay. The process of construction proved expensive here, due to the constant shifting of the tracks, but in this great desert country where water is scarce the costs are considered remarkable.

The inner face of the dam is paved to a depth of 2 ft. with basalt rock hauled for a distance of 1½ miles on a maximum 3 per cent grade from below the dam. This dam is 91 ft. high, its slope 2½ to 1, crest width 20 ft., length along the crest, 1523 ft., and volume 624,890 cu. yd. The outlet from the reservoir is through a 10-ft. circular tunnel operating under a 76-ft. maximum pressure head.

The total cost of the south dam was $268,095, the costs being distributed as indicated in Table 4.

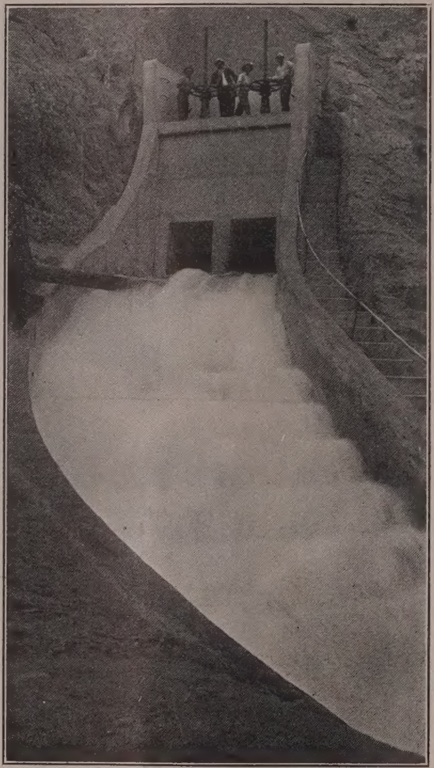
The Haiwee reservoir will supply the capacity of the aqueduct for a continuous flow of 80 days. The total cost per foot of waterway for direct field charges, including auxiliary construction, etc., was $13,726.

From the outlet tower there is a drop of 196 ft. in 2 miles to the Haiwee power site, where a maximum of 6200 hp of electrical energy can be developed.

COVERED CONDUIT

There are 13.148 miles of covered conduit in the Rose Valley division, the hydraulic slope being 0.0004; this section is also the same as the Freeman division conduit. The excavation in this division was done with model 40 Marion power and steam shovels, equipped with a 25-ft. boom and a ¾-yd. bucket. The shovels worked to a 72-ft. radius which consequently controlled the location of this class of work. The normal width of excavation was 12½ ft., with a depth of 10½ ft.

A flat-slab cover 6 in. thick at the sides and 8 in. in the center, reinforced with ⅝-in. square twisted rods, spaced 18 in. apart, and Clinton triangular mesh was used. The rods were placed below the mesh and 1 in. from the bottom of the slab. Where the top cut was from 3 to 4 ft., rods were spaced 9 in. apart, and where the top cut was in excess of 6 ft. rods were spaced 6 in. apart.



First Water Passing Cascade Drop

All concrete-lined work is plastered in order to obtain the most favorable coefficients of flow. Plastering cost from 1½ to 2 cents per square foot for labor and material. The plaster in the same ratio of cement to sand as the mix of concrete is found to give the best results, as the concrete and plaster then maintain the same rate of expansion and contraction and consequently the plastering does not scale off.

TABLE 4—COST OF SOUTH HAIWEE DAM

	Quantity, yds.	Unit cost	Total cost
Miscellaneous expense	623,747	$0.014	$9,928
Track	623,747	0.032	19,965
Core-wall excavation	27,032	1.04	28,130
Core-wall timbering	27,032	1.53	41,550
Core-wall puddle	27,032	0.287	7,170
Main fill	598,747	0 248	148,870
Riprap	16,110	0.70	11,268
Total direct charge			$266,881

Average total cost of dam per cubic yard of volume, 42.8 cents. To this add 32½ per cent for equipment, road, trails, pipe line, buildings and field overhead; and 4 per cent for general office and executive administration.

The 9 miles of work in the South Antelope Valley, done under contract, cost the city $14.30 per lineal foot, while similar work done by force account cost but $11.10, a saving of $3.20 per lineal foot.

TUNNELS

There are 42.903 miles of tunnels on the aqueduct, exclusive of the 8.8 miles of power tunnels; about 15 miles of tunnels were added to the original location as construction proceeded. With the exception of the Elizabeth tunnel, which was started in October, 1907, the tunnels on the Jawbone division were begun in October, 1908. Bids were asked for the construction of the Jawbone division, but were all rejected, for the board considered them excessive. The actual field costs in this division, completed, were $750,000 under the lowest bid received.

There are 164 tunnels on the aqueduct, most of which were driven from two or more headings, and there were no appreciable errors in grade or alignment on the entire project. The tunnel of greatest length is the Elizabeth tunnel, 5.09 miles in length.

Between the Jawbone tunnels side-hill conduit and flumes were used to cross the small canyons between portals, while in the latter locations the policy was to push the line farther back into the main ridge, driving the tunnels from adits and eliminating the expensive construction of flumes, culverts and side-hill work. Flumes cost in excess of $50 per foot, while the tunnel costs but $30, besides being the most satisfactory type of construction and most easily maintained. In the boring of the tunnels the Leyner air-hammer type of machine drills predominated. Fourteen Ingersoll-Rand cross-compound air compressors were used on the project. Muck was handled by General Electric and Westinghouse locomotives.

All tunnel work on the aqueduct was done under a bonus system and was a great success; rapid mucking was accomplished by this system. The tunnel foreman, shift boss, miners and muckers shared the bonus. All back-trimming was done by the crew sharing the bonus. The over-backage was about 30 per cent. The bonus varied from 20 to 40 cents per foot over the base rate, depending upon the class of material encountered.

TUNNELING RECORDS

A standard tunnel crew consisted of 16 men in untimbered tunnels and 23 men in timbered tunnels. In dry tunnels miners received $3 per day and muckers and timbermen $2.50 per day, and in wet tunnels 50 cents per day more. The American record was broken four times on the Elizabeth tunnel, the maximum progress being 604 ft. in a 30-day month. This tunnel was completed Feb. 28, 1911, fifteen months ahead of estimate. The average daily progress was 10.8 ft. per day for each heading.

In tunnel 17M, a 2-mile tunnel in the Jawbone division, the world's record of 1061 ft. for soft rock tunnels was made.

On the aqueduct 1,500,000 bbl. of cement were used; 900,000 bbl. were manufactured by

TABLE 5—ELIZABETH TUNNEL COSTS, PER LINEAL FOOT

Equipment	$6.50
Pipe lines and operation	.46
Road, trails and maintenance	1.40
Buildings	1.49
Surveys	.24
Miscellaneous losses, tests, phone, operation, etc.	.53
Division administration	2.12
Average cost direct charge	51.10
Executive administration	1.92
Grand total	$65.76

the city's Portland cement mill at Monolith; 300,000 bbl. by the city's Tufa cement mills at Fairmount and Haiwee; and about 300,000 bbl. were purchased from the California Portland cement mills. This cement compares favorably with any Portland cement purchased or manufactured in California. The Monolith mill was in charge of Mr. Roderick Mackay, chief mechanical constructor of the aqueduct.

The Fairmount reservoir is located at the North end of Elizabeth tunnel and 136 miles south of Haiwee, the maximum water surface being at El. 3035, making a drop of 535 ft. from the Haiwee gateway or the beginning of the covered section of the aqueduct. The capacity of this reservoir is 7620 acre-ft., with a surface area of 165 acres. Its purpose is to store water chiefly for power. The dam, of the hydraulic fill type, will be 115 ft. high.

TABLE 6—DIRECT FIELD CHARGES FOR DRY CANYON RESERVOIR

Core wall excavation and tunnels, 5350 cu. yd., at $4.07	$21,710
Core wall fill and puddle, 5350 cu. yd. at $0.73	3,902
Dry fill, 95,390 cu. yd. at $0.26	25,207
Hydraulic fill, 45,610 cu. yd. at $0.176	8,038
Drain pipe	5,291
Spillway	1,715
Paving 1 ft. thick, 470 cu. yd. at $2.95	1,385
Paving, oil	1,225
Miscellaneous expenditures, 146,350 cu. yd. at $0.07	10,181
Total cost of dam in place	$84,046

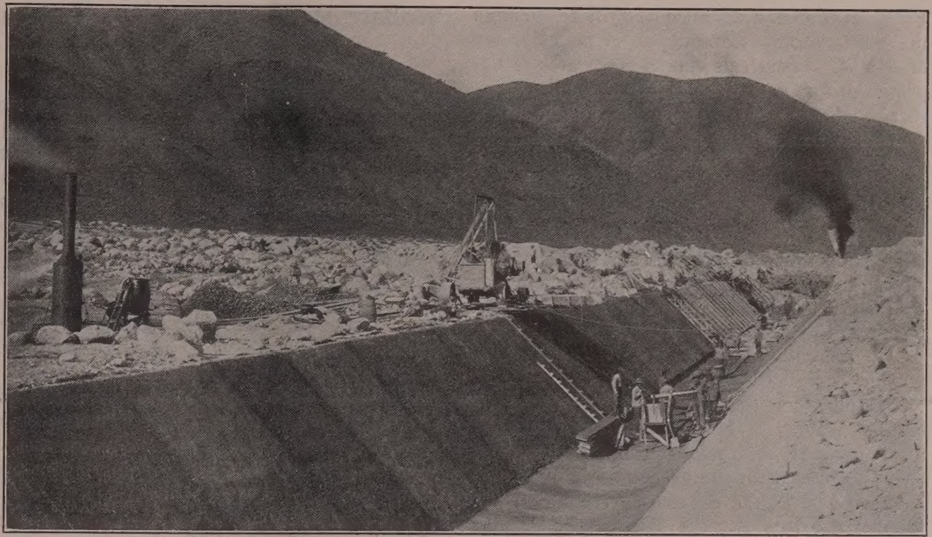
Average total cost per yard in dam, 57½ cents. To this should be added 20 per cent for equipment roads, field, overhead, etc., and 4 per cent for general office and executive.

DRY CANYON RESERVOIR

The Dry Canyon reservoir is for the regulation of the fluctuating flow through the power plant back to normal. It is located at the south portal of tunnel 77 of the Saugus division, with a capacity of 1325 acre-ft. The hydraulic fill dam is 528 ft. long and 61 ft. high.

The outlet from this reservoir is by an automatic floating weir. It is a sheet iron tank, 30 ft. in diameter, the drum being 4 ft. thick and 12 ft. high, operating around a circular concrete tower so as to rise and fall with the water surface like a gas tank.

The direct field charges for the Dry Canyon Reservoir are given in Table 6.



Mixing Plant and Forms for Lining Canal with Concrete in Olancha Division

The Fernando reservoir with a capacity of 23,000 acre-ft. is about 1 mile below the south portal of tunnel 108, the end of the aqueduct proper. The dam is being built under the supervision of the Bureau of Waterworks. It is 140 ft. in height, the side slopes are 2½ to 1, and the length along the crest is 2060 ft. Of the total volume of 2,675,000 cu. yd. 1,450,000 yd. were in place on Oct. 1. It is a hydraulic fill dam. At the present time 4500 cu. yd. per day are being placed at a cost of 7 to 8 cents per yard, while the wagon haul for building up the toes is costing 17 cents per yard. This dam will be completed during the summer of 1914.

SIPHONS

The specifications for boiler plate steel as recommended by the American Society for Testing Materials were accepted and used by the aqueduct officials for the construction of all steel plate siphons. There are located on the aqueduct 9.39 miles of steel siphons and 2.14 miles of concrete siphons. Their construction was described in the Engineering Record of July 19, 1913, page 60.

At the south portal of tunnel 108 in the Saugus division at an elevation of 1461.23 ft. is the end of the aqueduct proper. From this point for a distance of 24 miles to the city mains the work is being carried on by the water department.

The entire work has been completed attended by only one failure, that of the Sand Canyon pressure tunnel, which failed while under test during the latter part of April of this year. This failure has been freely discussed in a former issue of this publication. This pressure tunnel has been replaced by an all steel siphon which was completed Sept. 15 and tested on Sept. 20.

Table 7 gives the cost per foot of constructed waterway of the aqueduct by divisions, and by classification of structures.

TABLE 7—COST OF AQUEDUCT WATERWAY PER LINEAL FOOT, BY DIVISIONS

Division	Total direct charge, auxiliary construction and field overhead	Total field plus general office and executive
Owens Valley	$2.690	$2.969
Alabama Hills	10.858	11.563
Olancha	9.512	10.119
Haiwee	13.726	14.547
Rose Valley	10.340	11.05
Little Lake	31.71	33.66
Grapevine	28.52	30.26
Freeman	11.73	12.48
Jawbone	26.78	28.43
Mojave	11.10	11.82
N. Antelope	16.34	17.45
S. Antelope	17.50	18.77
Fairmount	47.74	50.61
Elizabeth	63.71	67.84
Saugus	28.40	28.59

	Direct field charge	Total direct plus all auxiliary expense
Unlined canal	$1.79	$2.83
Lined canal	8.38	11.15
Covered conduit	10.50	13.85
Tunnels	27.23	36.71
Siphons	25.82	34.29
Flumes	42.50	57.95
Reservoirs	12.20	17.14

Add 4 per cent for general office administration.

WAGES

The minimum wage paid on the work was $2.25 per day of 8 hours and the average $2.75. Approximately $12,000,000 was expended for labor and $7,000,000 for rights-of-way and lands, and $1,500,000 for freight, livestock, electrical energy, etc.

From the end of the aqueduct on the north side of the Fernando Valley the water drops 160 ft. in a cascade water-fall and travels a distance of 2 miles through a conduit to a spillway and tower in Fernando reservoir. The total drop is 326 ft. At El. 1135 the water is diverted into the reservoir and into the Fernando siphon crossing the valley for a distance of 12 miles through 72 to 62-in. steel pipe. Thence the flow is through the

Constructing Conduit on Steep Sidehill in Grapevine Division

Franklin tunnel, a distance of 3781 ft. through the Santa Monica Mountains, at an elevation of 850 ft. From this tunnel the water is bypassed down the canyon to the Franklin reservoir, a distance of 7600 ft. The maximum surface water elevation is 575 ft. and the reservoir capacity is 1100 acre-ft. The water is diverted from the reservoir for a distance of 3800 ft. through a 68-in. pipe. From here it is carried through a 40-in. pipe for a distance of 6 miles to the western limits of the city and connected into the distributing mains. This work is well under way and will be completed in the early part of the spring of 1914.

SALE OF POWER

The plan for the sale and distribution of surplus waters is largely problematical, but from general discussions the district plan seems to meet with the greatest approval, and the Board of Public Service by resolution has adopted the rate for the sale of the surplus water at 1 cent per miner's inch per hour, or a cost of $11.68 per acre per year—1 in. to 7½ acres—making the total annual cost to the consumer for one miner's inch $87.60.

Engineering Record References to Los Angeles Aqueduct Articles

Title	Page	Date
Great Waterworks Project of Los Angeles	488	Nov. 3, 1906
Owens River Aqueduct for Los Angeles Water Supply	107	Jan. 26, 1907
Water-Supply System of Los Angeles	228	Feb. 29, 1908
Construction Work on Aqueduct	393	Apr. 3, 1909
Rapid Tunnel Driving (letter)	28	July 10, 1909
Public Water Supplies (editorial)	311	Sept. 17, 1910
Cement Mill	331	Sept. 17, 1910
Tapering Surge Chamber	188	Aug. 12, 1911
Location and Design of Conduits	716	Dec. 16, 1911
Construction	6	Jan. 6, 1912
Difficult Tunneling	71	Jan. 20, 1912
Driving the Elizabeth Lake Tunnel	72	Jan. 20, 1912
Administration Charges	96	Jan. 27, 1912
South Haiwee Earth Dam (editorial)	115	Feb. 3, 1912
South Haiwee Earth Dam	116	Feb. 3, 1912
Water Power	126	Feb. 3, 1912
Injuries to Workers	131	Feb. 3, 1912
City Underground Maps (editorial)	450	Apr. 27, 1912
Concrete Flume and Culverts	698	June 22, 1912
Inverted Siphons	722	June 29, 1912
A Commemorative Monument	395	Oct. 12, 1912
Subterranean Water Storage	687	Dec. 21, 1912
Steel Pipe Siphon	16	Jan. 4, 1913
Heavy Sidehill Excavation	48	Jan. 11, 1913
Wages	95	Jan. 25, 1913
Failure of Pressure Siphon in Rock (editorial)	623	June 7, 1913
Failure of a Concrete-Lined Siphon in Massive Granite	640	June 7, 1913
Antelope Valley Siphon	60	July 19, 1913
Los Angeles Aqueduct Power (editorial)	197	Aug. 23, 1913
San Francisquito Power Station No. 1	218	Aug. 23, 1913
Surge Tanks on Penstocks (editorial)	281	Sept. 13, 1913
Surge Tank at San Francisquito Power Station No. 1	299	Sept. 13, 1913
Butterfly Gates for the San Francisquito Power Station No. 1	327	Sept. 20, 1913
Large Tank at San Francisquito Power Station No. 1 (letter)	420	Oct. 10, 1913

The local system has been taxed to its very limit during the three summer months just past, the daily consumption jumping from 55,000,000 gal. per day to an excess of 80,000,000 gal. For a continued heavy draft due to a period of excessive heat the present system is at its maximum. The timely completion of the Los Angeles Aqueduct has insured the steady future growth of the city of Los Angeles and its suburbs.

The completion of the construction of this great water supply system is a lasting monument to its designer and chief engineer, Mr. William Mulholland, who was ably assisted by Mr. J. B. Lippincott, assistant chief engineer.

A NEW EUROPEAN RECORD in tunneling was made last August in the building of the Swiss Hauenstein tunnel, according to "Wasser und Wegebau Zeitschrift." The previous record was set in July, 1909, when 1013.5 ft. were broken out in a large Alpine tunnel. The present record shows 1051.9 ft., thus exceeding the previous one by 38.4 ft.

Wallace Report on Chicago Railroad Terminals

Declaring that the traffic of the railroads of Chicago is constantly outgrowing their facilities for carrying on the business and also that the traffic of the city has outgrown the capacity of the streets in the central business district, Mr. John F. Wallace, in his recent report on railway terminals to the city council committee, considered the problem as a whole and endeavored to balance the functions of the different parts so that when readjusted the result will be a present-and-future convenient, efficient and economical system of transportation for both the city and the railways. As noted in the Current News Section last week Mr. Wallace recommended that the present site be retained for the new Union Station.

Commenting on the intense concentration of business activity within the ¼ sq. mile of restricted area north of Van Buren Street, he stated that the condition so urgently demands relief that immediate action is of vital importance. The outlet for expansion of the business district south of Twelfth Street should not, in his opinion, be barred by the proposed location of terminals occupying the space between State Street and the river. On the west side from Twelfth Street north the railways are limited to a narrow strip along the river, and since the tracks are so much below the street grade as not to interfere with east-and-west viaducts, it is deemed proper to leave this railway strip to such intensive development as the roads choose to make.

PRESENT CONDITIONS

As to the life of the present stations the report states that three of the five older stations have served their purpose and must soon be replaced with more commodious structures. The La Salle Street Station, by the growth of business, will require enlargement or replacement within the next ten years. The Grand Central Station, while admirably arranged, is badly located, with a drawbridge across the Chicago River near its train entrance. It is also of small capacity. The Union Station Company has submitted plans for a greatly-enlarged passenger terminal located on Canal Street, upon substantially the same site as to trainshed and tracks that it now occupies. The Dearborn Station Company has not as yet submitted plans for a new and enlarged passenger terminal. The Illinois Central has plans in course of preparation for a new and greatly enlarged station, located at Twelfth Street and the lake front, with a possible capacity on two levels for taking care of all the railroads on the south side of the city now using the stations east of the Chicago River.

Measured by the needs of the community alone, at least two general locations for railway passenger terminals are necessary to meet present requirements—one on the south side and the other on the west side, between Van Buren and Madison Streets and adjacent to Canal Street.

As the new Chicago & North Western Station at Canal and Madison Streets has been designed to serve the purposes of one road, another station to provide for the group of roads now established in that locality and using the old Union Station becomes a necessity, as much from the standpoint of the public using it and the business interests situated in the heart of the city as from a railroad point of view.

The Illinois Central has signified its willingness to make provision in its new station, on reasonable terms, for the passenger service of all the south-side roads; and with its main lines extending south and west to intercept readily the traffic of these roads, there appears to be no substantial reason why the grouping of the passenger terminals of the south-side lines at Twelfth Street and the lake front is not only feasible and practicable, but in every way advantageous to the railways, to the city, and to the community in general. The elimination in the course of time of three of the existing passenger terminals—the Dearborn, the La Salle and the Grand Central—would dispense with unnecessary stations and simplify the railroad situation within the business district.

TWO CENTERS PROPOSED

With two permanent centers for the development of the railway passenger terminals of the city thus established—the one at Twelfth Street and the lake front, and the other near Canal and Madison Streets—the distance between these terminals would be approximately 1 mile east and west and 1 mile north and south, the locations being at the northwest and the southeast corners of a district 1 mile square, within which would lie the central business district of the city. To complete ultimately the system of passenger intercommunication between the railways, and to make their terminals conveniently accessible from the outlying portions of the city and suburbs, a connecting railway system could be provided of dimensions sufficient to permit the movement of standard railway passenger equipment directly between the terminal stations, with surface outlets suitably arranged at convenient points for rail connection with the tracks of the terminal stations.

A further use of such an inner circle of communication between the railway terminals could be the extension of the suburban train service of all the roads of the city to include the circuit of the terminal stations and of the business district, thus putting every part of the city and its suburbs in direct connection with every railway radiating from the city. The difficulties in the way of practically carrying out this idea at the present time are so apparent that it is suggested only as a general plan that would seem desirable to keep in view in future work.

NEW UNION STATION

The plan of the new Union Station terminal provides for the station building, a monumental structure, to be located on the west side of Canal Street, occupying the entire block between Canal, Clinton and Adams Streets and Jackson Boulevard, with a passage the full width of the block under Canal Street to the concourse and trainshed, between Canal Street and the river.

The plan of the 2430-ft. trainshed provides for a double stub layout of tracks, with a passenger concourse 300 ft. wide, extending entirely across the trainshed between the two sets of station tracks, the level of the concourse and of the connecting platforms to be upon the same level as the floors of the cars. East of the trainshed and outside of it through tracks connect the north and south ends of the terminal. The track layout at the north end will consist of ten stub tracks, varying in length from 980 to 1460 ft., aggregating 11,840 lin. ft. available for trains, with four main-line approach tracks; at the south end there will be fifteen stub tracks, varying in length from 1060 to 1410 ft., aggregating 18,080 lin. ft. available for trains, with six main-line approach tracks. It is estimated that in functional activity the new station will have four times the capacity of the old one.

A particular merit of the design of this station, according to Mr. Wallace, is the arrangement of the trainshed on the plan of stub tracks, with a broad passenger concourse and platforms all on a level with the car floors. This plan offers to the public the most simple and direct method of approach and departure, and is the most convenient method by which passengers can go to and from trains without climbing stairs or using long ramps.

Concerning the comparative advantages or disadvantages of a central union station for all Chicago railways, as against independent stations or separate groups of stations situated around the business center, Mr. Wallace states that he cannot see the advantage of the former either in convenience to the public or in economy or convenience of operation to the railroads; in fact, experience in other localities would seem to show, as far as the railroads are concerned, an increase in fixed charges and expenses of operation entirely out of proportion to any material advantages gained thereby.

Bound up with the passenger terminal situation is that of the freight terminal problem, at least in the case of the Union Station, which in its proposed form overlaps present freight facilities to the south. The city council was asked to pass an ordinance for a freight terminal, with elevated approaches, extending southward from Van Buren Street between Jefferson and Desplaines Streets. Mr. Wallace disapproved of this plan and recommended depressing the incoming tracks and forcing the north frontage southward to Harrison Street to permit a westward extension of the business district by opening Congress Street to the west side.

Danger Signals for the Color-Blind

A new type of colored danger signal for use in industrial plants is advocated by Dr. Francis D. Patterson, director of the department of sanitation and accident prevention of Harrison Bros. & Company, Inc., of Philadelphia. The design of the signal is a blue circle within a wide yellow rim. Dr. Patterson states that of the 4 per cent of all men who are defective in color perception the majority cannot distinguish red. He brings the further indictments, in seeming contradiction of railroad-signal tradition, that red is evanescent and possesses relatively little luminosity and that it fades rapidly.

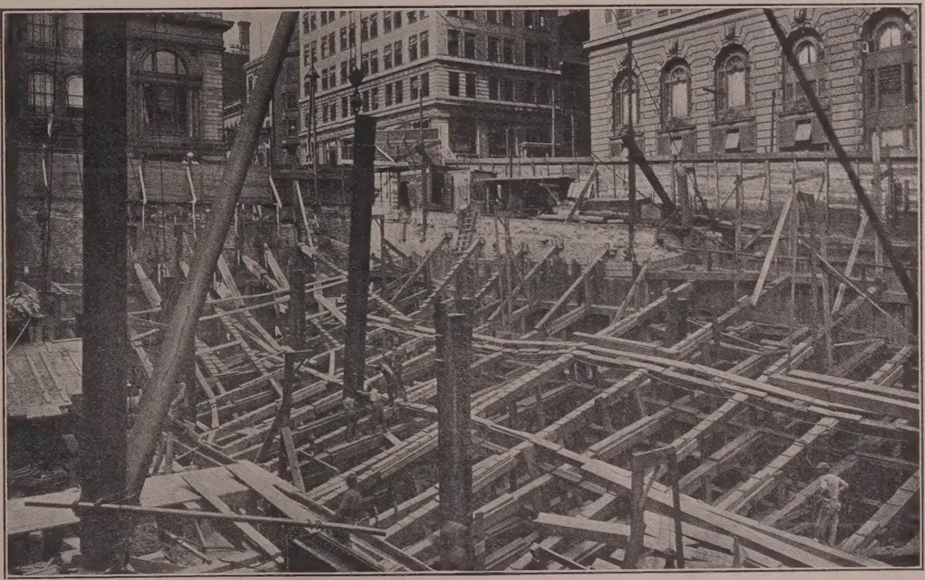
Dr. Patterson's arguments in favor of the blue-yellow combination were deduced from experiments. First, he states, since yellow and blue are the only colors which excite a normal color sensation as soon as they become visible as colored, they can be seen at a greater distance in daylight and are more easily distinguishable in a poor light than any other color or combination of colors. Further, all other colors appear either yellow or blue until they pass within the yellow blue area of the eyes, when they are recognized. Yellow and blue are also the most luminous colors of the spectrum and are more permanent than red. Finally, in the rare instances in which workmen are unable to distinguish yellow and blue, these colors will appear red or green to them, affording a protection a red signal could not give.

The new signal is discussed in its application to industrial buildings only, where the color-blindness restrictions existing in railroad and marine service would cause grave and unnecessary hardship.

Deep Excavations and Underpinning in Dry Sand

The 100 x 150-ft. steel frame office building for the Union Central Life Insurance Company at Fourth Avenue and Dye Street, Cincinnati, is 270 ft. high to the top of the main roof and 554 ft. high to the top of the tower. It has forty-seven main columns with loads up to 3,300,000 lb., supported on I-beam grillages with concrete footings from 46 to 54 ft. below the surface of the street resting on coarse dry sand and loaded to 11,000 lb. per square foot, as described in the Engineering Record of Sept. 21, 1912, page 328.

The site was once occupied by a post-office building and subsequently by the five-story granite building of the Chamber of Commerce which has recently been destroyed by fire, leaving the heavy walls standing and part of the roof in position. All of the floors had fallen into the basement and it was necessary first to support the roof on falsework built up from the basement. Steel stiff-leg derricks were then installed and took down the old wall masonry, including many 7 and 8-ton ashlar stones.



View of Heavy Bracing and of Derricks Setting Columns between Struts

Underpinning with Sectional Wall

Adjacent to one side of the lot is the 150 x 26-ft. nine-story Mitchell building, with floors supported on brick walls having rubble footings on the sand 8 ft. below curb level. Before excavation was commenced adjacent to it this wall was needled and underpinned by a new foundation carried down to the gravel stratum below the foundation for the new building. Beginning in the middle and at the ends of the wall 6 x 6-ft. pits were excavated to a depth of about 50 ft. and sheeted with 3 x 8-in. square-edged yellow-pine planks, 5 ft. long, braced with 4 x 6-in. rangers 3 ft. apart vertically, the ground being firm enough to stand alone while the excavation was being carried down 5 ft. farther and the new set of sheeting installed. In each well two men excavated the earth by hand and loaded it into ½-yd. buckets hoisted by two men on top. The gang of four excavated each well in an average time of three days. After the completion of the wells forms were built in them for the face of the 4-ft. foundation wall, which was built of 1:2:4 concrete mixed in a Smith machine located on the lot. Concrete was delivered from the machine in wheelbarrows and dumped by means of chutes into the pits.

The pits were excavated and concreted in successive sets of three until the complete new foundation wall was built to within 3 ft. of the bottom of the old wall and three continuous staggered rows of 5-ton jackscrews were set on it engaging the old masonry, which was lifted from the needlebeams and maintained on the new wall until the latter had been thoroughly loaded and adjusted to take up any settlement, after which the weight was transferred to wedging blocks, the underpinning being then completed in the usual manner.

Deep Excavation

Simultaneously with the underpinning the general excavation was made on the opposite side of the lot, which was isolated from the underpinning by a row of 4-in. wooden sheet piles 20 ft. long, driven across the lot on the center line. The outer side of the lot was also inclosed by sheet piles driven with mauls as the excavation progressed and heavily braced with longitudinal and transverse horizontal timbers, successive tiers of which were installed as the work progressed and supported by vertical shores at intersections. The excavation was made by hand, and the spoil, shoveled into buckets, was hoisted by two guyed derricks which commanded the entire area of the lot and dumped them into 12-yd. hopper bottom storage bins, from which wagons in the street were loaded.

General Excavation

The general excavation was carried down to a depth of 42 ft. below the curb. Some of the foundations were carried 12 ft. deeper. Care was taken to locate the horizontal bracing to clear the columns; but the pit was so deep and the timbering so heavy that considerable difficulty was experienced in lowering and setting the heavy foundation girders, some of which were 43 ft. long and weighed 26 tons. As three sides of the lot were bounded by streets, the danger of injury to adjacent structures was considerably reduced. The absence of ground water was an additional advantage for the contractor, who carried the excavation down through sand to the required depth without accident.

Mr. Cass Gilbert and Messrs. Garber & Woodward were the architects. The Thompson-Starrett Company, of New York, was the general contractor.

Rock Drilling in the Tennessee River

About 211 miles below Chattanooga is located the Tuscumbia bar, extending approximately 2 miles down the river. It is composed of a series of blue flint-limestone ledges overlaid with a thin coating of gravel. The project for improving this stretch contemplated the excavation of a channel 150 ft. wide and 5 ft. deep near the south shore, connecting the deep water above and below the obstruction. As the material to be removed was a very hard flint rock, it was found necessary to drill and blast it before it could be dredged. In the "Professional Memoirs" of the Corps of Engineers for September-October, 1913, Mr. J. E. Hall, assistant engineer, describes this work and has incorporated in the article cost data of unusual value to the contracting engineer. These cost data are included in the following abstract.

Extensive drilling being necessary in rock on top of which the depth of water varied from a few inches up to 5 ft., it was thought advisable to mount tripod drills on floats having sufficient buoyancy to carry men, drills and material to operate them and still be able to float in shallow water.

The floats used for the first season's drilling (1911) were composed of nine small boats, the dimensions of each being: Depth, 1 ft.; width, 5 ft.; and length, 25 ft. These were arranged in three rows, with three boats in each row, and a space of 2 ft. was left between the lines of boats through which to operate the drills. During the first season they were turned longitudinally across the current and the float was held in place with spuds 6 in. square. When the boats were so arranged the floats were 19 x 75 ft. At each placing of the drill unit two lines of holes were drilled, extending halfway across the channel, and twelve holes were put down in each line. These lines being 7 ft. apart made the average spacing of the holes 6¼ x 7 ft., and the holes were put down to a depth of 7 ft. below low water.

Drilling Progress

The drilling was carried on on this basis until Dec. 15, 1911, when it was terminated by high water and bad weather. During this season's work about 2500 lin. ft. of channel, beginning at the upper end of the work, were drilled and blasted. Where the detonation was perfect the rock was usually broken up, but often in such large pieces as to be very difficult to handle, and it was sometimes necessary to break them again with mud-capped shots before they could be handled.

Table 1 shows the 1911 season's work, giving the unit cost of drilling for each month, also the stage of the river, depth of water over the rock drilled and depth of the drilling in the rock.

Table 1—Results and Cost of Work during 1911

Month	Lin. ft.	Average depth of holes, ft.	Gage, ft.	Depth of water over rock, ft.	Cost	Unit cost
August	3,520	4 to 5	.4 to 2	3 to 5	$1,336.60	$0.38
September ..	6,338	3 to 5	.3 to 1.5	3 to 4	2,871.20	.40
October	6,921	3 to 5	1 to 5	3 to 7	3,450.00	.50
November ...	10,373	4 to 5	1 to 3	2 to 6	4,634.39	.43
December ..	1,679	4 to 6	2 to 12	4 to 7	1,007.40	.66
Total	28,831				$13,299.59	

Average unit cost for season 1911, $0.46. Number of linear feet of channel drilled, 2,500.

Extensive repairs being necessary to the floats before entering on another season's work, it was decided to rearrange the boats composing them, turning them lengthwise with the current and leaving the opening between the boats 1 ft. wide. The floats used in the season of 1912 were composed of thirteen boats, 1 x 5 x 25 ft. long, put together in this way, making the spacing of the lines 6 ft. apart instead of 7, as in the previous year. The dimensions of this float were 25 x 77 ft.

At each placing of the drill unit or float twelve lines of holes were drilled, each line having six holes. As the length of the boat was 25 ft., the spacing of the holes was made 4 x 6 ft. and the holes were put down to a depth of 9 ft. below low water. In addition to the advantage of narrowing up the spacing, the placing of the boats lengthwise with the current was an advantage, in that they were less affected by the action of the current and drilling could be carried on at a higher stage of water.

Table 2 shows the work during the season of 1912, giving unit cost for each month, stage of river, depth of water over rock, and depth of holes drilled.

Table 2—Results and Cost of Work during 1912

Month	Lin. ft.	Average depth of holes, ft.	Gage, ft.	Depth of water over rock, ft.	Cost	Unit cost
June......	3,720	7 to 8	3 to 5	3 to 5	$1,523.19	$0.41
July......	3,256	7 to 8	3 to 6	3 to 6	1,465.00	.45
Aug......	15,120	7 to 8	2 to 3	2 to 4	5,727.28	.38
Sept......	15,752	6 to 7	1 to 3	1 to 5	6,301.85	.40
Oct.......	20,250	5 to 7	1 to 2	2 to 4	9,317.50	.46
Nov......	20,042	5 to 7	.5 to 1.5	3 to 4	10,462.89	.52
Dec......	7,568	5 to 7	1 to 6	3 to 7	6,659.84	.88
Total...	85,708				$41,457.55	

Average unit cost for season 1912, $0.483. Number of linear feet of channel drilled, 2,500.

In comparing this season's work with that of 1911 it will be noted that the unit cost of the drilling is 2 3/10 cents in excess of the 1911 cost. This may be due to considerable advantage in weather and river conditions in 1911, and also to the fact that the drills were all new in 1911, while in 1912, especially near the close of the season's work, a number of them were badly worn and would not deliver a normal stroke. After September the cost is also augmented by the shortening of the days, making it necessary to work a longer number of hours at night. While the plants are very well lighted, quite a falling off is noted in their progress when comparing the result of an hour's work at night with an hour's work in the day. Early in October a number of the best drill men left the work to go back to their old stations at the furnace, and it was necessary to fill their places with new men.

Arrangement of Work

The following method was universally followed during both season's work: The drill floats were placed in proper position for work by accurately lining them with ranges, marking the center of each half of the channel, and by cross-ranges which marked the lower extremity of the drilling and blasting already done. A float thus lined is in position for drilling one-half the width of the channel for a distance of 25 ft., or the length of the boats. At each setting twelve lines of holes were drilled, with six holes in each line. Each float carried eight drills, all of which are operated by steam.

To prevent the holes from filling with gravel and silt the drilling was done through tubing or pipe 3 to 4 in. in diameter. From three to five drill bits were used in putting down each hole, the first bit being 2½ in. in diameter and the last 1¾ in. When the hole was down to a proper length, a pipe that would exactly fit the top section of the hole was put in, the bit taken out and the hole loaded, the charge consisting of 80 per cent gelatin dynamite and varying from six to ten sticks, according to the depth in the rock. The stick having the primer was placed about one-third of the way down from the top, having two or three sticks on top of it and from four to six under it. The charge was firmly packed down in the bottom of the hole with wooden poles which fitted very closely the section of the hole. When the loading was completed the hole was marked by a cane, which was firmly embedded in the charge, leaving the top about 6 in. above the surface of the water, and the primer wire was looped around the top of the cane. When all the holes were loaded the primers were all carefully connected so as to make three circuits, twenty-four holes to each circuit, leaving the end wires of the first and fourth line looped around the top of the cane so that they might be readily found and connected with the lead. The float was then dropped down from over the holes and a set of lead wires attached to each circuit. When these were connected (insulated tape being used for making all these connections) the float and tender were dropped back about 250 ft. below and the charges ignited by using three large batteries.

During the first season the drills were all sharpened by hand, but in 1912 a Leyner drill sharpener, operated with compressed air, was installed. A considerable saving was effected, as one blacksmith and helper were able to sharpen steel for twenty-four drills, while three blacksmiths and helpers were necessary to do this work by hand.

The seeming excessive cost of drilling here is due to the character of the rock, which is the very hardest of flint. It was found a very difficult matter to get steel that would stand this rock. The slightest mistake in tempering would cause the bits to fail at once.

Comparison of Work

The season's work for 1912 covered the heaviest part of the blasting, as it began at the upper end of the extremely shallow water and extended entirely below it. While the number of linear feet of holes drilled in 1912 was about three times the amount drilled in 1911, only about the same channel area was covered, viz., 2500 lin. ft. This was on account of the narrow spacing of the holes, the additional depth drilled and the fact that this season's drilling covered the heaviest part of the work. There is now left above the dam an area undrilled about equivalent to 1200 lin. ft. of channel.

The average day's work for one drill unit operated with a double crew was 432 lin ft.; average hour's work, 27 lin. ft.; average hour for one drill, 3 3/10 lin. ft.

Table 3—Cost of Tender Equipped for Work

Building hull, cost of labor and subsistence....	$1,346.66
Lumber and iron, nails, spikes, oakum, etc......	1,902.40
Brownell boiler of 90 hp....................	900.00
Leyner drill sharpener, No. 2..................	697.10
Compressor for running same..................	533.00
Receiver for air storage......................	64.80
Steam capstan.............................	495.00
Setting up and connecting the above-named machinery	185.40
Building one-story cabin for sheltering machinery, material, and labor..................	465.00
Total cost of tender......................	$6,589.36
Type of float now in use, composed of 13 boats; cost, labor and material...................	1,140.00
Eight drills, E-24 Ingersoll-Sergeant...........	1,950.00
Steam hose, drill steel, iron pipe..............	365.00
Cost of float, equipped for drilling...........	$3,455.00
Cost of tender............................	6,589.36
Cost of one drill unit......................	$10,044.36

During the first season's work any boat having sufficient boiler power that could be spared from the plant was used as tender. The floats could be very quickly built, and they were put to work in this way pending the building of a suitable tender. The type of boat built for this purpose is a barge 30 x 80 x 4 ft., provided with three spuds. It is equipped with a 90-hp boiler, a Leyner drill sharpener, a compressor for operating same, and a steam capstan for handling the barge. The sharpener is driven by air, the compressor in use being one manufactured by the Chicago Pneumatic Tool Company, having a 9-in. cylinder for steam and air, with 11-in. piston stroke, and a piston displacement of 130 cu. ft. of air per minute at 100-lb. pressure.

Three drill units were operated during the season of 1912, and it was found that one drill sharpener could keep steel in shape for the three units, each of which carried eight drills. Only one tender, as described in the foregoing, has been built for the work here, the other two units being furnished with steam by spare pieces from the plant.

During 1911 28,831 lin. ft. of holes were put down, loosening up about 23,155 cu. yd. of material, costing as follows:

Actual field cost, including material, salaries subsistence, etc.	$13,299.59
Deterioration of plant on account of season's work	3,840.00
Overhead charges	664.98
Total cost	$17,804.57
Cost per linear foot	0.617
Cost per cubic yard, loosened	0.76

During the season of 1912 85,708 lin. ft. of holes were drilled and blasted, loosening up 76,185 cu. yd. of material estimated in place, costing as follows:

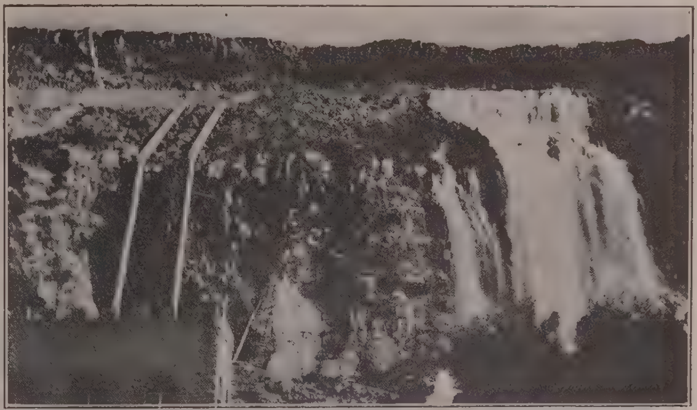
Entire field cost, material, etc.	$41,457.75
Estimated deterioration of plant	7,500.00
Overhead charges	2,072.88
Total cost	$51,030.63
Cost per linear foot, drilled and blasted	.595
Cost per cubic yard, loosened	.677
Total amount of material loosened up in the two seasons, cu. yd.	99,566
Total amount of dynamite used, 75,450 lb.	$12,855.36
Amount of dynamite per cubic yard, ¾ lb.	.127

Thousand Springs Power Plant

A unique water power is the one which has been developed during the past year at the famous Thousand Springs in southwestern Idaho. The springs are located on the north end of the Snake River, about 8 miles from the town of Wendell. They have for years been a favorite resort for picnic parties and sightseers.

A large part of the State of Idaho is overlaid with a great lava sheet, which covers the sedimentary rock below to a depth of 100 ft. or more. At the point in question the Snake River channel is about 300 ft. deep and its north bank shows a line of contact between the lava and the sedimentary rock at a height of about 180 ft. above river level. Water from a large drainage area to the north and east finds its way through the more or less porous and fissured lava until it reaches the impervious rock below, and then along the plane of contact in the direction of its dip, until it finally breaks out of the river bank in the form of innumerable springs.

Architectural Treatment of Thousand Springs Station



Relation of Penstocks to Canal and Spillway

These Thousand Springs extend for a distance of about 1400 ft. along the contact line and are so close together and so large as to present an appearance of a waterfall of great beauty. The exact source of the water is unknown. It may come from all parts of the drainage area or it may include the waters from one or more of the lost rivers in Idaho.

The stream has an average flow of about 750 sec.-ft. and is quite uniform in temperature and quantity, varying little during the seasons of the year.

Several unsuccessful attempts have been made during the past twenty or thirty years to develop the power of this falling water. The method of collecting the water has been the chief problem and various plans have been proposed. One company started to drive a series of tunnels into the bluff along the contact plane. It was thought by means of crosscuts all of the water could be intercepted and brought out through the tunnels. Work on this plan was abandoned, however, after the driving of a few short tunnels.

In the fall of 1911 the present Thousand Springs Power Company was organized. Surveys and a study of the general conditions at the springs were initiated to determine the best method of making the development, and excavation was commenced a few days later. The plant went into commercial operation on Nov. 1, 1912.

General Plan

The plan adopted for collecting the water and delivering it for use at the power house is as follows: A concrete wall, varying in height from 3 to 16 ft. and in thickness from 2 to 8 ft., was built along the outer edge of the impervious rock shelf. After excavating the covering of broken lava and earth back for a distance of from 15 to 20 ft. so as to form a canal back of the wall at a point above the power house site, the shelf was cleared to a width of about 40 ft. to form a small forebay, from which steel pipe lines carry the water through the concrete wall and down the cliff to the power house at its base.

The initial installation was for 3000 hp only, but the design of the forebay wall and spillways is such that the complete development of about 10,000 hp can be made by simply continuing the canal and wall so as to collect all of the water and by adding additional pipe lines and machinery without disturbing the original installation.

The wall for the first installation is 400 ft. long and collects approximately one-third of the water. That this method of gathering the water is the correct one is proved by the fact that the cliff below the wall shows no signs of leakage, the water being sealed off completely. The construction of this wall was

Spring Water Collector, with Spillway in Rear

Discharging Spring Water during Canal Construction made rather difficult owing to the provision which had to be made for handling the water. There was, of course, no way to stop the water from entering the space back of the wall, so it had to be diverted from the new work by means of sandbag dams placed just back of the wall forms and caused to pass out through sluice gates in the finished portion.

On account of the constant temperature of the water, which is about 60 deg. Fahr., no provision had to be made for handling ice, and the intake consists merely of a simple screen and a cast-iron headgate in front of the opening to each of the two steel pipes. No gate valves are used at the turbines, but the headgates are so arranged that they may be quickly closed from the power house by means of a tripping rope.

Equipment

The present equipment of machinery consists of two 1500-hp Pelton Francis type turbines, direct-connected to Westinghouse alternators. The power-house building is made of the lava rock picked up in the immediate vicinity and is designed to resemble a mediæval castle, with tower, minarets and battlements.

The high tension apparatus, including 44,000-volt transformers, disconnecting switches and electrolytic lightning arresters, are all mounted out of doors. The transformers are equipped with wheels and stand on steel rails so that they may be rolled upon a low truck provided for the purpose and then rolled into the power house, where they may be picked apart by the crane whenever it should be found necessary to make inspection or repairs. The placing of this high-tension apparatus out of doors removes the most dangerous elements from the power house and effects a considerable saving in the cost of installation.

The entire output of the plant is sold at the transformer terminals under contract to the Great Shoshone & Twin Falls Water Power Company. Work on the final development has now been begun. The canal is being extended so as to collect all of the water.

The Thousand Springs Power Company is composed of Salt Lake and Denver capitalists. Mr. John Dern, of Salt Lake City, is president, and Mr. Lafayette Hanchett is general manager. The plant was designed and built under the supervision of Mr. Owen H. Gray, consulting engineer.

Hauling a 160-Ton Span Out of Water 36 Feet Deep

Temporary Structures for Supporting Railroad Tracks

Considering the constant necessity for the rebuilding of railroad bridges and culverts under traffic, and the ever-growing importance of grade-crossing elimination, the methods of supporting tracks to permit construction work under them merit much study. Taking care of traffic while bridges are being built to eliminate grade crossings will be the subject for investigation during the coming year of a committee of the American Railway Bridge and Building Association. For the past year a committee has been studying the general subject of temporary support for tracks, and while it is true that each case must be settled more or less on its merits, the committee has formulated the following general rules, which the association has adopted:

Piles should be driven before stringers are put in, as they can be driven so that they will be directly under the stringers and will not have to be pulled or jacked into line; the fill under the track is not badly disturbed when putting in caps, only a small amount of blocking is needed to keep the track safe for full speed of trains at all times, and the work of placing stringers is made easier by having caps in place to slide the stringers on.

Usually in cases where there is a choice of piles, framed bents or crib work, piles should be used. In places where the bottom is soft or uncertain, or where there is the slightest chance for scour by water, piles should always be used, driven, if possible, at least 4 ft. below the bottom of the proposed excavation.

Where local conditions permit, the span length should be the same as the standard trestle span, which will permit of using stringers from stock, and in many cases second-hand material can be used. On spans longer than safe timber stringer construction I-beams or girders should be used.

Whenever stringers only are used they should be supported on cross-sills at the ends, except when the width of excavation is to be small compared to the length of stringer. As a general rule the depth which can be excavated is such that a 1½ : 1 slope from each side of the bottom of the excavation will not undermine the sills under the ends of the stringers. This cannot be given as a rule always to be followed, as in some soils a 1:1 slope would be safe and in others a 2:1 slope would be unsafe.

In general piles, being safer, should be used rather than framed bents or grillage structures.

A detour trestle should be built when a suitable alignment of track can be obtained and the cost of this method will not be materially more than of a trestle in place.

Bridges should be rolled in when conditions will permit, and the expense is not materially greater. In cases where it is not possible to roll in and the new bridge must be rebuilt in place, the old metal floor system should be left in whenever possible, and the falsework should be designed with this in view.

Where the bottom is uneven short sills should be used when possible, unless the bottom is rock, in which case the sills may be omitted entirely; in either of these cases care should be taken to have all bents well braced, with crossbraces as low as possible. The bracing should conform to the standard plan as far as possible.

Wooden scab plates, securely spiked or bolted, should be used on temporary work; runner timbers or girts should be used in all cases at every break in the uprights.

Recovering Submerged Bridge

The Pennsylvania Railroad Company's Franklin bridge over the Shenango River, between New Castle and Franklin, Pa., was washed away in the flood of Jan. 28 and 29, 1913. A 160-ton through-plate girder 111½ ft. long fell into the river, with one end on the bottom in water 36 ft. deep and the other end projecting about 18 ft. above the surface of the river.

The span was removed from the water by the Ketler-Elliott Erection Company, of Chicago, who used one 75-ton derrick car of its own, one 120-ton and one 75-ton wrecking car belonging to the railroad company. The span was moved entire through a considerable vertical and longitudinal distance by two twelve-part tackles made with five-sheave blocks and single-sheave snatch blocks rove with 1400 ft. lengths of Roebling ⅝-in. steel cable. Horizontal piles laid transversely on longitudinal skids formed a sort of roller platform to receive the bottom flanges of the shore ends of the girders as they were dragged ashore. The work was done under an agreement that the Erie railroad tracks on which the span was first landed should not be obstructed more than two hours.

Detroit River Tunnel

The Detroit River tunnel, built by sinking floating sections of the tubes and connecting them under water, was referred to on page 284 of the Engineering Record of Sept. 13, in connection with the sinking of the Harlem River tubes. It was then stated that the method used at Detroit was due to Mr. Olaf Hoff. While this is true as to the contractors' methods, the plan originally was suggested by Mr. William J. Wilgus, then vice-president and chief engineer of the New York Central & Hudson River Railroad and chairman of the advisory board of engineers as to the tunnel work. In fact, the tunnel company agreed to protect the contractors against any possible claims for infringement in view of the origination of the plan, or method of construction, by the tunnel company's engineers.

Design of Steel Mill Buildings—Part II

Practical Considerations Influencing Type and Proportions of Members, Commercial Features, Detailing, Architecture, Drawings and Specifications

By R. Fleming, Designing Engineer, American Bridge Company, New York City

In proportioning the members a copy of some standard general specifications for steelwork will call attention to many points which might otherwise be passed by. Thus, the length of compression members shall not exceed 125 times, and those used for wind and lateral bracing 150 times, their least radius of gyration; material less than ¼-in. thick, except for fillers, shall not be used, and a number of similar requirements will be found.

The designer of a structure may not make the working drawings, but he should be competent to do so. Unless he has had actual experience in detailing it is probable his design will have more or less serious defects. For this reason one of our largest structural-steel companies makes it a rule that engineers before entering their designing department should have not less than three years experience making shop drawings.

A common error is to so proportion a girder that a sufficient number of rivets cannot be used to develop the flange stresses. Crane girders have been designed with the flanges of 6 x 4-in. angles (4-in. legs vertical) and cover plates, ample for compression or tension with a web sufficiently thick for shearing stresses. When it comes to be detailed it is found impossible to get enough rivets in the 4-in. leg of the angles to transmit the stresses from the flanges into the web plate. A thicker web plate to increase the bearing values of rivets or, better still, 6 x 6-in. angles allowing a double row of rivets might have obviated the difficulty.

Commercial Features

In addition to the purely technical there are also the commercial features of structural engineering. Some of these it is well for the designer to know. For instance, the handbooks of the steel companies generally classify the sections as standard or special. The latter, on account of the smaller demand, are rolled less frequently, and hence it is often difficult to obtain them in a reasonable length of time. When the demands for any given size have accumulated to a certain tonnage it is included in the schedule of rollings. A whole structure has been held up indefinitely waiting for some pieces of an odd size. As a rule, the fewer the sizes used the more prompt will be the delivery.

Three or four years ago the engineer of a building near Philadelphia used a ship channel that he had found listed in an old handbook for the tracks of an extensive system of suspended trolleys. After the contract was placed it was found these channels had not been rolled for five years. There was not enough tonnage to warrant a special rolling; besides, the rolls were out of order. With considerable annoyance the engineer changed his tracks and track supports.

Mere ingenuity or originality of design is not enough to warrant a departure from standard types of trusses, girders, or other parts unless accompanied by a reduction of cost. Precedents need not be followed blindly, but "indifferent originality is more expensive and less creditable than good imitation, and the success of the engineer is directly proportional to the success of his work and to the lowness of its cost."

The pitch of the roof will largely determine the outline of the roof truss. The Fink type of truss is the one most commonly used for roofs of 5 or 6-in. pitch. The Warren and Pratt trusses are well adapted to a roof nearly flat. The factors determining the most economic spacing of wall-bearing roof trusses are the trusses themselves, the bracing, and the purlins. If the steel columns, side girts, or crane girders are used, they also are to be considered together with the cost of foundations. It is to be noted that the net cost per pound of purlins is much less than that of columns and trusses.

The theoretical economic spacing of roof trusses is about one-quarter the span, but the actual spacing may be governed entirely by conditions peculiar to the structure under consideration. If the covering or sheathing rests on purlins a spacing of about 16 ft. is commonly used up to spans of 65 ft.; beyond that the spacing may be one-quarter the span. Where plank sheathing rests directly on the trusses the spacing may be 8 to 10 ft. for 2-in. and 12 to 14 ft. for 3-in. tongue-and-groove plank. This method can often be used to advantage for spans up to 40 or 50 ft. As there are no purlins in this case, there should be a system of bracing to hold the trusses in place.

If spans are of unusual length or roof loads are very heavy it may be found economical to use jack rafters resting on latticed or beam girders framed into the roof trusses. Purlins can be used on top of the jack rafters or wood sheathing can extend from rafter to rafter.

Wooden purlins used with steel trusses are often made to act as struts for a system of diagonal bracing, but the practice is not to be commended. The struts as well as the diagonals should be of steel.

Roof trusses are mostly made up of angles, sometimes entirely so. Flat bars should not be used for tension members. They are easily bent in handling and the stress coming upon them is often not enough to straighten them. If there are loads between panel points producing a bending stress sufficient to warrant it, the top chord should be made of two channels or two angles and a stem plate. The stem plate should not be too thin for its width or it will buckle along its lower edge when fully loaded. The bottom chord may be of similar construction where bending stresses occur from various causes. It is a good plan to detail the lower chord channels about 1 in. apart so that ¾ or ⅞-in. bolts can be run between them and attachments easily made. Purlins should be made of single rolled shapes—channels, beams, or angles. Trussed purlins should not be used.

Columns

The plate and four-angle type of column with cover plates when necessary is mostly used in mill buildings on account of its low cost of fabrication and the ease with which trusses, girts, or beams can be connected to it. For interior columns, where the loads and unsupported distances of columns are not great, the steel section known as Carnegie mine timber can be used to advantage. The H columns rolled by the Bethlehem Steel Company have been received with favor, but are used more in office buildings than in mill buildings. The box section of columns generally made up of two channels latticed is not so well adapted to mill buildings on account of its higher cost and the greater difficulty of painting and inspection. Connections are not so easily made, and it is likely to become a receptacle of dust and moisture. Closed-box sections should not be used at all.

The exterior columns should be proportioned for (1) direct stress due to roof, floor, crane, or other loads; (2) bending due to wind on the sides of the building; and (3) stress due to eccentric loads. For columns supporting crane girders, the flange of the column under the girder is usually considered as carrying the total crane load together with its share of roof load and any floor load that may come upon it. When this load is large, or the unsupported distance of column is great, a channel or plate can be used on the backs of the angles under the crane girder. Of course, bending stresses from the wind or from eccentric loads can be lessened by increasing the width of the columns. When the columns are assumed to have their lower ends fixed, care should be taken that the bases are firmly anchored to the foundation by set bolts.

Kneebraces

Attention is here called to that important member, the kneebrace. While it should be of abundant strength to take the stresses coming upon it, it is folly to make the kneebrace "good and strong" unless there is provision to take the thrust or tension at each end. If the trusses and columns have been figured for uniform load only, the stresses due to the kneebraces should be determined and sections increased where necessary. Similar calculations should be made when traveling cranes require gusset plates to be used in place of kneebraces.

The sight is not altogether unfamiliar of a kneebrace of liberal proportions fastened at one end with three or four rivets or bolts to a latticed column and fastened at the other end to the bottom chord of a roof truss having at that point a diagonal member consisting of a single light angle. Such connections are able to develop the strength of only a very light kneebrace. The lacings in the columns should be replaced by a plate, the number of rivets in the connections should be increased and the single-angle diagonal in the truss should be made of two angles. In a cement mill near Allentown, Pa., are some kneebraces of 6 x 6-in. angles fastened by only a single bolt at each end.

Beams and Girders

The safe loads that can be carried by the various sizes of rolled beams are given in the Carnegie and other handbooks. Care must be taken that the tabular safe loads are reduced when the top flanges are without lateral support for more than 20 times their width. In certain cases, for instance where a ceiling is supported by beams, the limits for deflection should not be exceeded. The comparative cost of rolled beams and plate girders should be considered when there is a choice between them. Preference is generally given to rolled beams where they can be used. In proportioning plate girders their depth should be taken as the distance between the center of gravity lines of the flanges, with one-eighth of the web area allowed as part of the area of each flange. A more nearly correct way is to design plate girders by their moments of inertia.

For lateral stiffness the top flange of a crane girder should not be less in width than 1/20, or 1/25 at the most, of its span. A channel riveted to the top-flange angles gives stiffness and adds to the flange area. If the rolled channels are not of sufficient width a built channel of a plate and two angles answers the same purpose. Additional area if required can be made up of cover plates. If runway girders are riveted to the sides of columns it will be difficult to secure a proper alignment. They should rest either on the tops of columns or columns should be stepped off to receive them. For light cranes they are sometimes carried on brackets attached to the columns. Crane rails are sometimes riveted to the top flanges of girders, but this allows no alignment. The rails should be fastened by hook bolts or by adjustable clamps and bolts.

An end greatly desired in all steel structures is that they should be well braced. No general rules can be given for bracing. It is not entirely a matter of calculation. Experience, common sense, and a comprehensive grasp of the situation are needed. In bracing for wind it should not be forgotten that the wind blows against the ends of a building as well as against the sides. Two years ago a pier shed along the New Jersey coast had to be braced longitudinally to keep the ends from blowing in. The designer had made no provision whatever for wind pressure against the ends of the building. A certain amount of bracing is needed for erection purposes. Better reduce the main members themselves than economize in bracing. A building seldom fails from lack of material in the main members, although these are sometimes greatly skimped.

Architectural Features and Details

Ruskin gave the advice, "Do not construct ornamentation, but ornament your construction." A mill building is designed, first of all, for utility, and the architectural features are usually a secondary matter. But this is no reason why a building should be an architectural monstrosity. A slight expense will oftentimes change a structure that is unsightly to one that will at least not be an eyesore. In the future there will be greater economic values attached to symmetry and beauty than at present.

The part of the detailer in making or marring a structure can hardly be overestimated. Some of the buildings of the Gary plant of the United States Steel Corporation are 1400 ft. long and more than 100 ft. high. The engineer under whose supervision the designs were made writes: "While the assumptions of loads and their application to structures of this class may not always have been entirely correct, I am satisfied that we have had more trouble and failures due to the lack of proper consideration of the details of this structure than to improper assumption of their loading."

On receipt of the design drawings the detailer should examine them carefully to ascertain if he has all the necessary information. As he proceeds with the working drawings he should take up with the designer any points that are not entirely clear. He had better ask six foolish questions than omit an important question that should be answered. At the same time he should report any faults he may discover; for instance, swinging sash that if open would be struck by a traveling crane in passing.

As soon as possible a plan should be prepared for the purchaser, showing the size and location of column bases and anchor bolts. This drawing is generally called a "foundation plan"; but this name is a misnomer, as it usually shows only the location of steelwork. The loads coming upon the foundations from the steel columns should be given. The drawing now gives enough information for the foundation to be designed by others, as it is seldom that the structural engineer is called upon to do this. A crane clearance diagram, if there are traveling cranes, should also be prepared at an early date and sent to the purchaser for the use of the crane maker.

Details should be designed for strength and economy, in accordance with shipping facilities, cost of transportation and ease of erection. Connections producing secondary stresses are sometimes used when, by a little ingenuity, they could be avoided. When there is a direct vertical load on a plate girder, such as the wheel load from a traveling crane, sufficient rivets should be provided throughout the top flange of the girder to take the resultant of the vertical load distributed over a given length and the difference in flange stresses. Economy will be gained by having as many duplicate parts as possible. Close fits are expensive, so also is needless refinement in workmanship.

Shipping Dimensions

The maximum dimensions of a piece that can be shipped on a car will sometimes be needed. The railroads charge for freight by the pound in carload lots, with a higher rate for less than full cars. There is a minimum weight for a carload lot. If this was 30,000 lb. and the car was loaded with 20,000 lb., the rate per pound would prevail. The lesser loading, from the nature of the pieces shipped, can not always be avoided, but the detailer can often lessen or increase the amount that can be loaded on a car by the location of his splices and his decisions regarding field and shop riveting.

The steamship companies have, for export work, a rate per cubic foot of space occupied and another rate per pound. The rate which will bring them the most money prevails. This is liable to be the rate per cubic foot. The remedy is for the designer to detail his work to be shipped "knocked down" as much as possible. An extra rate is charged if single pieces exceed certain lengths or weights. Some trusses for a light highway bridge were once shipped from an inland shop to the steamer riveted together. The steamer charges for freight were more than one-half the price received for the bridge.

Erection and Expansion Joints

Ease of erection should be borne in mind by the detailer. While it may lessen freight charges to ship a truss in many pieces, it will increase the cost of field riveting in erection. A study should be made to determine which had better be done. Every erector of steelwork who has been long in the business will recall details that have unnecessarily added hundreds of dollars to his expense account. Beams framed between girders striking stiffeners, girders and framed beams striking rivet heads in columns, insufficient clearances elsewhere, and too close fits are common difficulties in erection.

In continuous structures more than 300 ft. long expansion and contraction due to changes in temperature must be considered. A few years ago an outdoor crane runway in Ohio was built with each girder bolted to its neighbor for the entire length of 1500 ft. The first change of season played havoc with that runway and it had to be altered at considerable expense. A mill building is much more flexible in this respect than a bridge, and for the average mill building of medium size no provision need be made. Expansion in the direction of the width of a building will take care of itself; it is unusual lengths that need be considered.

If preliminary drawings only are wanted to give the prospective customer an idea of his building, a great deal of information can be omitted that is necessary on the finished design. It is a wise plan, however, to make even the preliminary drawings as full and complete as time and the circumstances will warrant. After a contract is taken the design drawings should be carefully checked and such additional information added as will enable the drawing room to make its working drawings expeditiously and with certainty as to the intent of the design. The revised design should be sent to the purchaser for his approval. All dimensions, distances and sizes of materials should be shown on the drawing. Descriptive matter properly belongs to the specifications, though notes on drawings should be freely used even if their purport is repeated in the specifications.

Checking Drawings

A careful check should be made by the designer after the drawing is entirely finished. Every discrepancy found and corrected will save time and trouble in correspondence later. It is annoying in laying out the work to find the distances between two fixed points given differently in different places on the design. Another common annoyance is to find that the sum of several intermediate distances does not equal the figure given for the total distance. Sometimes a necessary distance or angle is omitted.

All angles and all sides should be given in the survey of a plot of ground. Right angles should be given as 90 deg. and not left to be assumed by the detailer. A common cause of annoyance is to find that the sides and angles of the survey do not check with each other when a trigonometrical calculation is made. This can be avoided by the man who makes the survey checking his work before it leaves his hands. The points of the compass should be shown in order to refer easily to any side or end.

A cross-section should be taken through a definite line on the plan and so marked. "Cross-section on line A-A" means much more than "Cross-section through machine shop." As many sections as are necessary to make clear the intent of the design should be shown.

The location of doors and windows and side and end bracing should be shown on the side and end views. The plan should indicate clearly the roof bracing. The bracing in the planes of top and bottom chords can be shown on one plan by using full lines for one and broken lines for the other. Angles with unequal legs should be marked so that there is no doubt as to which leg is vertical and which outstanding.

Loads and stresses throughout, maximum bending moments and maximum shears in all girders, and in beams used as crane girders, should be given. It should be stated whether stresses are given in thousands of pounds or in tons. The omission of this once caused some trusses to be detailed for tons when the stresses were given in thousands of pounds. The detailer was a new man and had been accustomed to stresses given in tons; the designer had always done his thinking in thousands of pounds. Stresses are generally given in thousands of pounds with the + sign denoting a tensile stress and the — sign a compressive stress.

Avoid Ambiguity

An essential in the design drawings, in fact in all drawings, is clearness. On a certain design, underneath the diagram of crane wheels, was the notation "Maximum wheel load, 30,000 lb." with an arrow drawn to each wheel. This was a cause of confusion because it was not known whether the 30,000 lb. was the load on each wheel or on both wheels. The designer intended "Maximum load on each wheel, 30,000 lb.," and it should have so stated. On the cross-section of another design below the column base was written "Two 1¼-in. anchor bolts 4 ft. long," with an arrow drawn to each of two anchor bolts shown in elevation. It was not certain whether two or four anchor bolts were wanted for the column. A repetition of the note on each elevation would have left no cause for questioning the meaning of the legend.

Abbreviations should be used very sparingly. It should be borne in mind that the design drawings pass into the hands of non-technical men. The notation "No. 20 G.C.S. covering" may mean No. 20 galvanized corrugated steel covering and "Pl. G." may mean plate girder to the man who makes the drawings, but to some that are unfamiliar with his work they are meaningless. The force of an office in Philadelphia spent a good part of the forenoon questioning the meaning of the notation "D. & M. plank" on a drawing which had been received in the morning's mail. At noon the designer came along and gave the information that dressed and matched plank was intended.

SPECIFICATIONS

Clearness is an essential quality in specifications as well as on drawings. They should be so written that the structural-steel contractor should be able to tell from them just what his work includes. There should be a clear-cut division between his work and that of other contractors. Stair work, cast-iron door sills, steel sash and doors, gaspipe railing and similar work should be so described that they can be bid upon separately if desired. The structural-steel contractor should furnish all bolts or rivets connecting steelwork with steel, but should not be called upon to furnish bolts for connecting the work of other trades to steel. Bolts for fastening woodwork belong to the carpenter, mason's anchors should be furnished by the mason.

What are known as "blanket clauses" should not be used when writing specifications for steelwork. Architects are great sinners in this respect. These clauses are generally due to lack of study on the part of the writer in finding out just what is required and are inserted to provide, at the expense of the contractor, for any contingency that may arise. Nothing is gained by this method, for the bidder adds enough to his bid to cover all uncertain meanings of the specifications and the purchaser pays the bill. Occasionally the bidder is caught napping and this renders him even more cautious afterward.

The specifications for one of the West Point buildings called for the steel contractor to furnish "all lintels over openings as required." Under this clause he was made to furnish several tons of lintels over openings for heating and ventilating pipes. While the heating and ventilating plans did show openings, there were no lintels called for, neither were they shown on any steel plan. Hence they were not included in the estimate upon which the bid was based.

It is not necessary to repeat in the specification the dimensions, sizes, and weights that are shown on the drawings. In brief, the specifications for steelwork should cover (1) an introductory section indicating the scope of the work and referring to the drawings by number; (2) the quality of the material and the tests to be made; (3) the workmanship, shearing and bearing values for rivets and bolts, whether field connections are bolted or riveted and who furnishes and sets anchor bolts; (4) the approval of working drawings; (5) inspection; (6) the erection of the steelwork; (7) painting, both in shop and after erection; (8) any special requirements.

If there are no general specifications, the definition of party of the first part and party of the second part, the provision for settlement of disputes that may arise, the terms of payment, and similar matters of a legal nature should be embodied in the steel specifications.

FOUNDATIONS

The most important part of a building is its foundation. If this is not stable the whole building is affected. The inquiry for a prospective mill building generally specifies "to be built upon foundations furnished by the owners," and the structural engineer feels relieved that the responsibility for the design of the foundations does not rest upon him. At the same time, if trouble ever came from the foundation the probability is he would be put on the defensive to show that the steelwork was not at fault.

For the ordinary mill building the loads on the columns are comparatively small and the building is on firm ground. This explains why so many foundations proportioned in a haphazard way answer their purpose. There are cases, however, where this careless method will not answer and trained judgment as well as engineering skill will be required. The difficulty is not to provide unyielding foundations on a compressible soil, but to provide foundations that will settle uniformly throughout the building. When the loads are large there will be some settlement, and if this is much more in some parts of the building than in others there may be serious trouble. Floors will become uneven, crane runways will get out of alignment, shafting will not run smoothly and a dozen other difficulties will be incurred.

Knowing the bearing value of the soil and the loads to be carried, it would be a simple matter of division to find the required area of footing if the entire load was a dead load or if the live load bore the same ratio to the dead load in all columns. But the live loads are intermittent and constantly changing, while the proportion of the live to the dead loads is different for the various columns. In the design of the modern high office building this is a serious problem. While in a lesser degree the problem enters into mill buildings, it often needs be considered. This is the case in factory or other buildings where the walls are carried by the exterior columns. The dead load forms a large percentage of the total load on these columns, but the interior columns carry mostly a live load. In a machine shop with a center aisle served by a traveling crane with a gallery or two on each side, the percentage of live load on the interior columns is much greater than on the wall columns. Schneider's specifications on this subject are:

"The live loads on foundations shall be assumed to be the same as for the footings of columns. The areas of the bases of the foundations shall be proportioned for the dead load only. That foundation which receives the largest ratio of live to dead load shall be selected and proportioned for the combined dead and live loads. The dead load on this foundation shall be divided by the area thus found, and the reduced pressure per square foot shall be the permissible working pressure to be used for the dead load of all foundations."

The writer prefers in mill building the word "average" to "largest" in "that foundation which receives the largest ratio of live to dead load." But no definite rules can be given for the proportioning of foundation footings. They must be determined from a knowledge of the local conditions and from experience in that particular line of work. If the engineer has not had this experience he should have his work examined by one competent to pass upon it.

When the column loads together with the bearing capacity of the soil are such as to require large footing areas, grillages of steel beams embedded in concrete are often used. Heavy masonry with deep excavations and excessive steel or cast-iron bases is thus avoided. In proportioning these grillage beams the loading should be considered uniformly distributed over the length on which it is applied, the beams uniformly supported from below over their entire length, and maximum bending to occur at the center of the beam.

Attention is called in conclusion to Ketchum's "Mill Buildings," which has long maintained a reputation as the leading text book on the subject; Johnson, Bryan and Turneaure's "Modern Framed Structures;" Schneider's "General Specifications for Structural Work of Buildings," the standard specifications for steel buildings; Tyrrell's "Mill Buildings," Perrigo's "Modern Machine Shops;" Merriman and Jacoby's "Roofs and Bridges;" Simpson's "The Design and Construction of Industrial Buildings," published in the Engineering Record of May and June, 1909; Gerber's "Some Commercial Features of Structural Engineering," published in the "Proceedings" of the Engineers' Society of Western Pennsylvania, April, 1907; the files of the Engineering Record, and other technical papers for the last ten years.

Seventy Batches Per Hour

Rapid concrete mixing and placing has been accomplished in the construction of Section 2 of the Lexington Avenue Subway, New York. A model 59 Ransome mixer of 8 cu. ft. capacity, charged with a pivot hopper and operated by a General Electric motor, has been installed at the surface of the street on Broadway at Worth Street and was recently timed for several minutes at one batch per minute. The superintendent states that frequently 70 batches per hour are recorded.

The machine is a portable one, mounted with the motor on wheels, and occupies only a few square feet of space between the curb and the street car tracks. As little storage room is available in the vicinity, supplies are delivered by wagons daily or hourly as required. The cement bags are piled close to the machine and sand and gravel piles are maintained so near that little wheeling is required and their location is easily changed from time to time as the machine is moved to keep near the concreting point. The machine discharges into Ransome carts, several of which are provided to take the concrete a short distance and deliver it to chutes by which it is deposited directly in the forms in the subway trench.

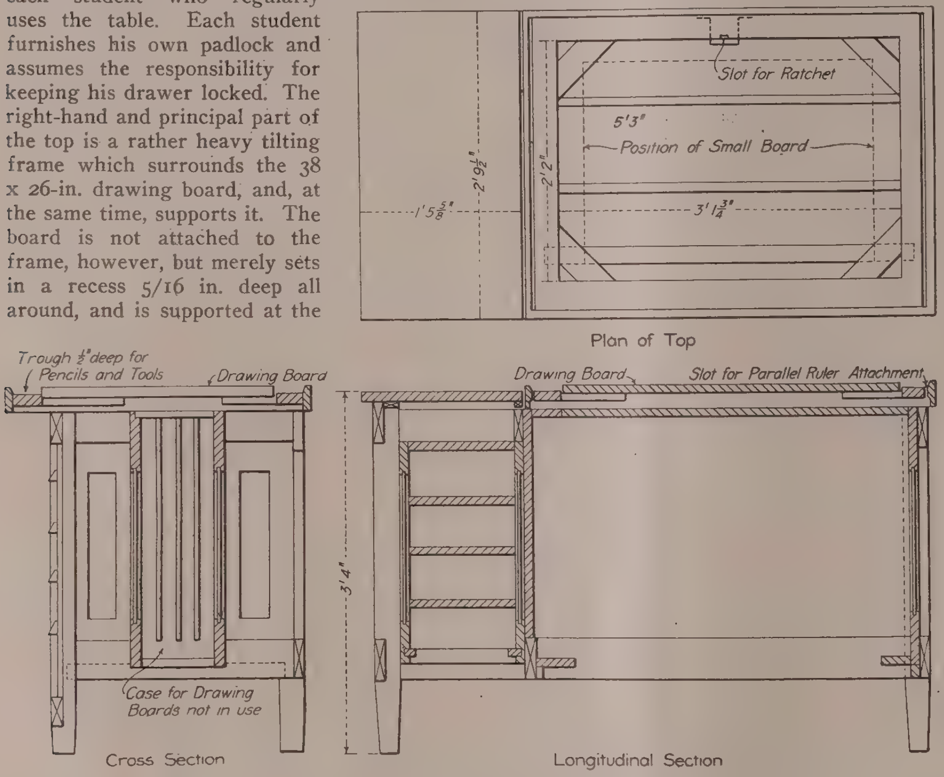
The mixing, at the rate above noted, is effected by a total force of 12 men; three of them wheel stone, one wheels sand, four help load the barrows, one handles cement bags, one is stationed on the discharge chute and one on the pivot hopper, besides which there is an extra man available for any required service. The contract section is a four-track structure about 2800 ft. long and involves about 30,000 yd. of concrete, as described in the Engineering Record of Dec. 14, 1912, page 660. The work is being executed by the Degnon Contracting Company.

Standard Drafting Table at the University of Illinois

By A. F. Comstock, Instructor in Railway Civil Engineering, University of Illinois

In 1910 the College of Engineering of the University of Illinois adopted a standard drafting table for use by the students. About fifty of these tables are now in service. The special features claimed for this table are stability, tilting top, interchangeable drawing boards, parallel ruler and general conveniences.

The table is built entirely of oak and has a pleasing appearance. Its general dimensions are: Length, 5 ft. 3 in.; width, 2 ft. 9½ in.; and height, 3 ft. 4 in. The supports are 3 x 3 in. vertical legs, connected near the bottom and top by horizontal braces, one of which also serves as a foot rest along the front side. On the left-hand side are four drawers, one for each student who regularly uses the table. Each student furnishes his own padlock and assumes the responsibility for keeping his drawer locked. The right-hand and principal part of the top is a rather heavy tilting frame which surrounds the 38 x 26-in. drawing board, and, at the same time, supports it. The board is not attached to the frame, however, but merely sets in a recess 5/16 in. deep all around, and is supported at the four corners only. This leaves the space under the drawing board entirely vacant except for the small corner supports already mentioned.



Sectional Views of University of Illinois Standard Drafting Table

The frame is hinged at the front so that it may be tilted to any angle desired. To tilt the top the draftsman reaches to the back of the frame, lifts the frame with one hand, while with the other he operates the prop or ratchet which holds the table at a given angle. The ratchet is of wood, hinged to the under side of the frame at the back.

Parallel-Ruler Device

A parallel ruler is attached to the tilting frame and takes the place of a T-square. It operates by a steel wire which passes around four pulleys connected to the under side of the frame at the corners. The ruler travels far enough to clear the board at both top and bottom. A ridge ½ in. high was left around the outside of the frame to prevent pencils and instruments from sliding. The drawing board is also arranged so that its top is ½ in. above the frame, thus forming a trough ½ in. deep by 3¾ in. wide all around the board. The trough along the front side is especially convenient for holding pens or similar instruments while drafting.

That part of the top to the left of the tilting frame and immediately above the drawers is stationary. Notebooks, triangles or other material may be left in this space. A vertical four-compartment case, accessible from the top, has been provided underneath the table for storing drawing boards. This arrangement permits each of four different students who use the table to have a separate drawing board, kept at its proper table, protected from dust and handling.

Rules Governing the Use of Heavy Vehicles on Highways

Rules for the use of heavy or fast-moving vehicles on improved roads are now the exception rather than the rule in at least the more thickly settled sections of the United States. It is unusual, therefore, to have their validity questioned. Yet the rules promulgated by Allegheny County, Pennsylvania, in 1912 were resisted in the courts and their legality has just now been sustained by the Superior Court of the State. An abstract of the rules and of the decision are given herewith.

Abstract of Rules

The first section of the Allegheny County regulations forbids transportation upon the improved highways by dragging or sledding (sleighing excepted), and the digging, cutting or injury of the surface of the road except by permission of the county road commissioner.

The next succeeding sections specify the widths of tires required for certain loads. Loads, including weight of vehicle, of 3500 lbs. or less must have tires at least 2½ in. wide. Loads between 3500 and 6000 lb. must have 4 in. tires, while loads between 6000 and 15,000 lb. must have 6 in. tires. Loads in excess of 15,000 lb. are not allowed on the roads without the permission of the road commissioner.

The use of a chain deadlock or steel strut with forked end to hold vehicles on grades is forbidden. When the use of a deadlock is necessary it shall be of a steel slipper or shoe design which meets the approval of the road commissioners.

Cleats or spikes on traction engines are forbidden unless they are rendered harmless by such filling as will oppose a smooth surface to the road.

The penalty for violating these rules is a fine which shall not exceed $100 for each offense.

Test Case

Suit for injury to one of the county's roads by the operation of a cleat-wheeled traction engine was brought against one Alvin Doughty in the county court and the defendant found guilty of two violations and fined $50 and costs for each. He appealed and the Superior Court has recently affirmed the judgment. In the opinion, furnished the Engineering Record through the courtesy of the road commissioner, Mr. John S. Gillespie, the Court, among other things, says:

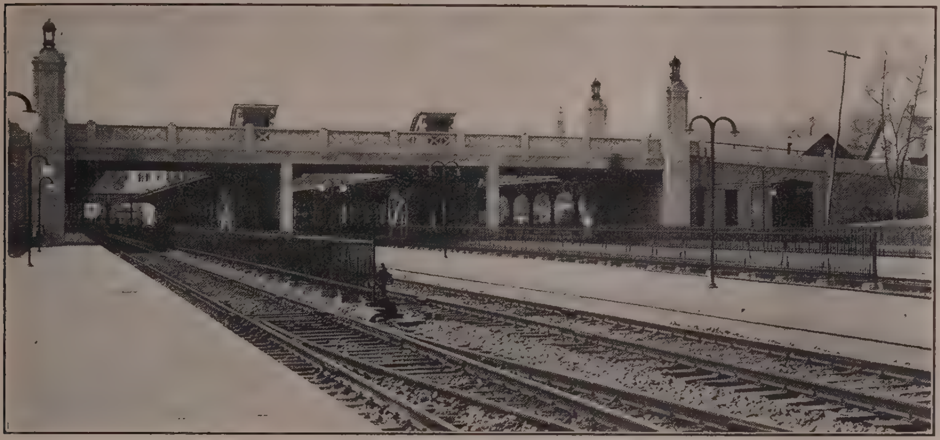
"Within the past quarter century the whole system of road making and maintenance has been changed to meet the demands of modern traffic; substantial and drained foundations are now universally specified with uniformly hard, smooth surfaces; all being done in the interest of durability, economy and to facilitate the movement of trade and travel.

"It appears from the record that there are about 400 miles of this style of road in Allegheny County, costing many millions of dollars. It cannot be denied that the cleated wheels of a heavy traction engine seriously indent and injure the surface of the improved roads; and in order to preserve them from present injury and ultimate destruction it is but reasonable and necessary that limitations and restrictions in regard to weight of load, style of wheel, tread, gage, etc., should be promulgated by the proper authorities. It is easy to conceive of an engine of such proportions and design that would permanently destroy any ordinary roadbed or bridge over which it would pass. If such a reasonable rule as is involved here causes a hardship or inconvenience to a few persons it may be a personal misfortune, but the benefits accruing to the general public in securing permanent and safe highways for public use are of much greater importance, and it would be unreasonable and inequitable to permit a very limited class of traffic to use the road in such a way as to destroy them for use by others.

Reasonable Use of Roads

The county is in a sense the public's trustee and holds these improved highways under the statute authority providing for their construction for the use of which they have been dedicated, just as a borough or city is a like trustee in the control of its streets. The right in the public to use the highways is the right to use them for the purpose of travel and commerce by any method not in itself calculated to prevent a reasonably safe use of the highway by others.

"There is nothing in the rules of the Allegheny County Commissioners to prohibit the use of the improved roads by traction engines. They only regulate by reasonable restrictions the manner of such use, and the testimony in this case shows that their requirement of a flat wheel instead of one with cleats or spikes is feasible and not unreasonably expensive. By observance of such requirements the traction engines may be used and the public highway is not endangered."



Grove Street Viaduct, with Ornamental Balustrade and Lamp Posts

Montclair Terminal Improvement on the Lackawanna

Replacement of Old and Cramped Facilities with Carefully and Artistically Designed Passenger Terminal and Separate Freight Layout

Montclair, N. J., is a city of about 25,000 inhabitants at the terminus of a 4-mile branch of the Delaware, Lackawanna & Western Railroad leaving the line through Newark and the Oranges at Newark. There are four other stations on the branch, all of which handle a large suburban business to and from New York, which is only 15 miles from Montclair. In addition to this the branch handles a heavy milk and ice business and serves a number of large industries along its route, particularly at Bloomfield.

Extensive improvements on the branch have been under way for the last three years. They consist of the elimination of twelve grade crossings, the construction of new and ornamental stations at Watsessing Avenue and Bloomfield, double tracking from Bloomfield to Montclair, sundry station improvements at Glen Ridge, and the entire reconstruction of the Montclair terminal. The early stages of the work were described in the Engineering Record for June 10, 1911; in this article a plan showed the location of the intersecting streets, and profiles showed the relation of the old and new grades as far as they were at that time established. Certain additional changes have been made in the profile between Bloomfield and Montclair and the work is now practically completed.

ARTISTIC FEATURES

In the entire work much attention has been given to the securing of artistic and pleasing effects. For bridges the following four types of design have been used: Steel bridges incased in concrete, elliptical arches of reinforced concrete, flat-topped reinforced-concrete girder-and-slab construction and thick reinforced slabs. In all structures a special effort was made in the design to relieve by architectural effects the plainness for which concrete is so often criticised.

Aside from the large increase in the capacity and convenience of the Montclair terminal, much attention has been given to the arrangement of buildings, driveways and grounds, and the terminal is believed to be one of the finest suburban terminals in the United States. It replaces what was admittedly a very unattractive layout.

Plans of the old and new layouts are shown for comparison. It will be seen that the old station was served by one track only. When a train had pulled into the station there was just room enough for the engine to uncouple, move ahead to the end of the track in Spring Street, and back down on the turntable. To accommodate the heavy early-morning business into New York it was necessary to store cars over night at the terminal, and this was inadequately provided for by the three stub tracks south of the station. Next to this was the 20 x 70-ft. freight house served by a single track, and beyond this were three short team tracks, which were obliged, with the freight-house track, to accommodate all of the local freight, milk and ice business. The rest of the railroad property between Grove Street and Bloomfield Avenue was useless, being occupied by Toney's Brook and a tangle of underbrush.

The throat of the yard was crossed at grade by Grove Street, and east of this point the narrowness of the right-of-way (66 ft.) compelled the company to confine itself to a small number of tracks. Pine and Bay Streets also crossed the tracks at grade within the limits of the projected revised terminal.

In planning the new terminal it was felt that all of the company's property west of Grove Street was needed for the passenger layout and approaches, and that an entire new location must be found for the freight business. The site chosen was that in the vicinity of Pine Street, between the existing tracks and Toney's Brook. This necessitated the acquisition of considerable land and the removal of a number of dwellings and other buildings.

The new station layout has six stub tracks, with the company's standard concrete bumpers at the ends. The concrete platforms are 650 ft. long, to accommodate ten 65-ft. coaches each. The tracks are on 13-ft. centers, and the island platforms are 17 ft. wide, the side platforms being somewhat narrower. The canopies extend as far as the Grove Street viaduct—somewhat more than half the length of the platforms.

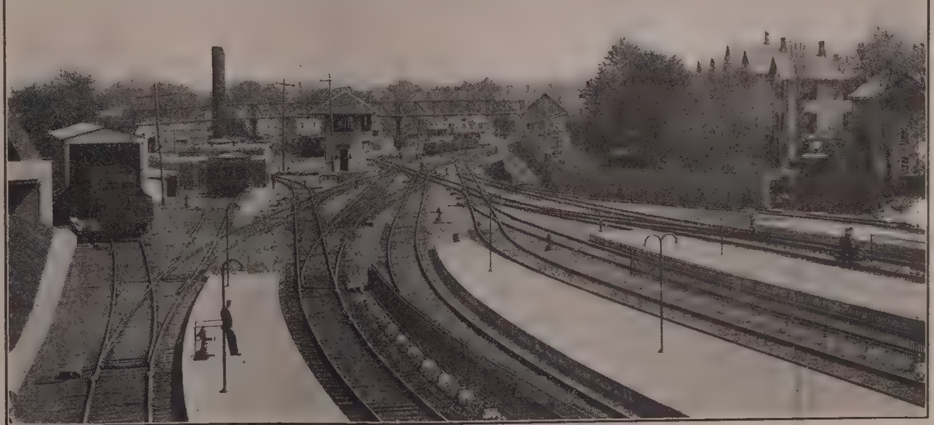
NEW STATION

The station is 45 x 125 ft. The walls are faced with tapestry brick, with trim of artificial stone, made of concrete surfaced with chips of the imported marble used in terrazzo floors. The main waiting room has a terrazzo floor, and its walls are faced with buff-colored pressed brick. The green glazed-tile roof is supported on ornamental steel arches. The driveways are paved with wood blocks on a concrete base, and as the illustrations show, much attention has been given to the details of park spaces, shrubbery and other ornamentation.

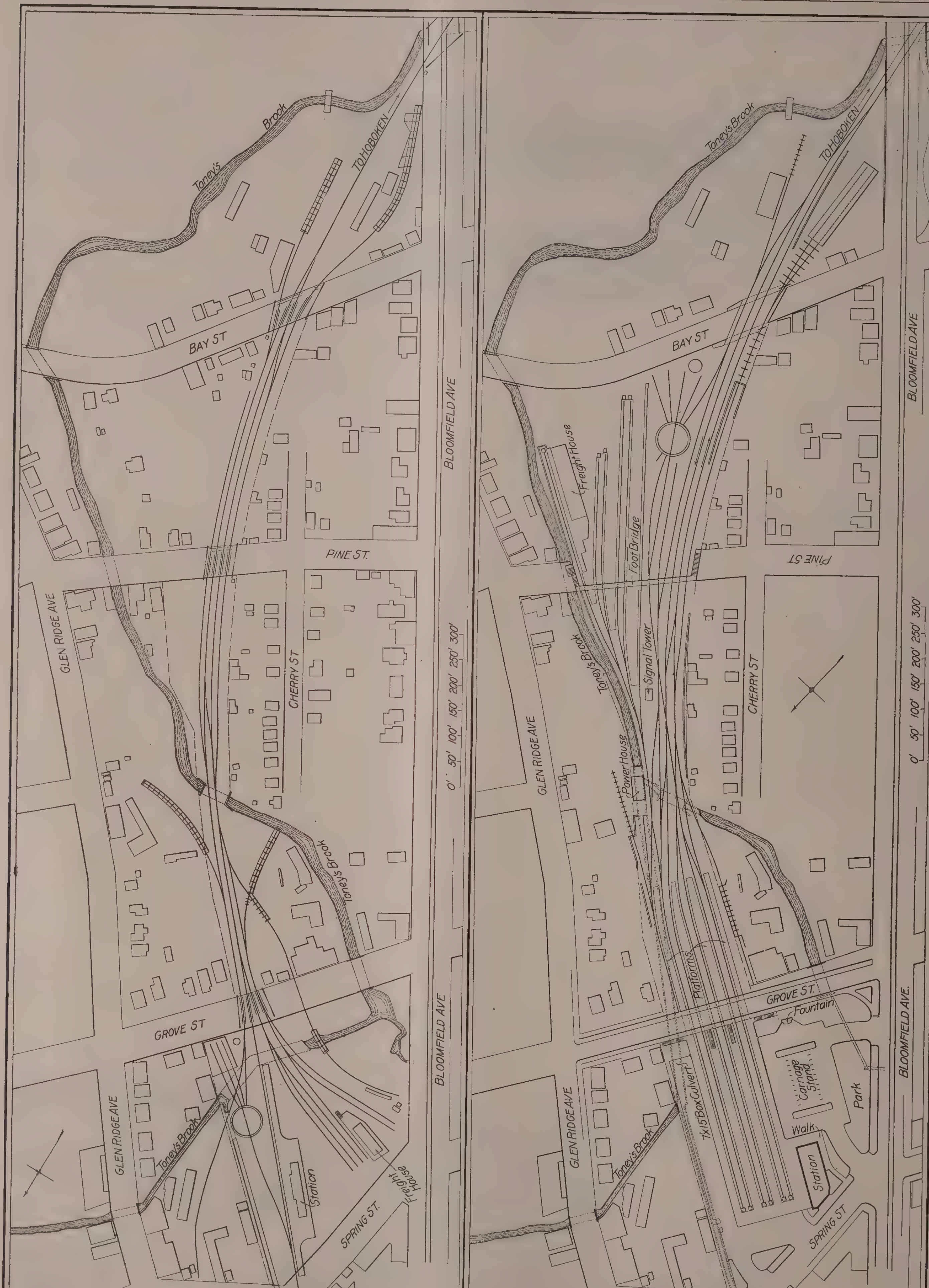
There is no heating plant in the station, all heat being brought from the power house 170 ft. beyond the end of the platform. The heat is conveyed in a pipe subway, which also carries all wires, passing along under the south platform to the ends of the tracks and then turning and running under the end platform to a point opposite the last track. A branch duct leads to the station, and piping reaches the end of each track, so that connections can be made and the cars stored over night alongside the platforms can be heated.

GROVE STREET VIADUCT

The ornamental Grove Street viaduct is of reinforced concrete, 50 ft. wide, with a 5.65 per cent approach grade from Bloomfield Avenue and a 6 per cent approach at the other end. In order to keep the south approach grade down and bring it out at Bloomfield Avenue and at the same time keep the underclearance above top of rail to the established minimum of 16 ft., the tracks were depressed about 2 ft. below the old grade. This still necessitated raising the Glen Ridge Avenue intersection about 2 ft. Structurally the viaduct is a right-angle crossing of standard girder-and-slab construction, with four reinforced-concrete stairways leading down to the platforms and station grounds. Attention is called, however, to the balusters, the ornamental lampposts and the kiosks over the stairways—all of concrete. The fills for the approaches are held with gravity walls with plumb faces and battered backs, surmounted



Throat of Yard, Power House, Signal Tower and Pine Street Footbridge



Comparative General Plans of the Layout at Montclair before Starting and after Completing the Terminal Improvement by a continuation of the reinforced-concrete ornamental baluster.

Toney's Brook Culvert

As the original plan shows, Toney's Brook crossed the right-of-way near Grove Street, where it was joined by a smaller stream running under Bloomfield Avenue, and crossed back again some 400 ft. east. Still another watercourse came down the east side of the right-of-way from Spring Street, and joined Toney's Brook near the turntable. The city proposes ultimately to cover this brook throughout the heart of the town, and in order to give it as nearly a straight run as possible and get it out of the way, it was decided to keep it on the north side of the right-of-way and carry it through by means of a 7 x 15-ft. reinforced box culvert. This culvert was designed for an earth foundation, with walls of four different thicknesses at various points. For instance, west of Grove Street, where the culvert carries only a roadway, the four walls are each 18 in. thick; on the other hand, under the Grove Street approach, and farther east, where the culvert is under a track, thicker walls are used, the side walls being slightly thinner than the top and bottom ones. In excavating for the culvert, however, rock was encountered for a large part of the way, and here the reinforced invert slab was eliminated and gravity side walls replaced the reinforced walls. The main culvert is 680 ft. long; it passes directly underneath the power house, coal pocket and service track, and, as already stated, excavation for it was largely through solid rock. The small watercourse is brought down by a 4 x 7½-ft. reinforced box with 10-in. walls throughout.

Freight Layout

Land had to be acquired and buildings removed on both sides of the right-of-way east of Grove Street in order to give room for the desired layout, and it was also necessary to relocate various tracks serving private industries, as shown. The greater part of the land required, however, was for the freight layout, all of which is on ground not previously owned by the company. Comparing the new layout with the old, the new freight house has three times the floor area, and has platform accommodations for six cars as compared with two for the old. The team tracks will accommodate about 51 cars, giving a total capacity for the freight layout of 57 cars as compared with about 20 on the old layout. The team tracks are on 12-ft. centers, with 24-ft. paved driveways between them.

No engine house has been provided in the new layout, for the reason that all details were developed with a view to future electrification of suburban service. A cinder pit has been built near the turntable, which is the old 65-ft. table moved to the new site, but engines required for the morning service from Montclair will be stored over night in the open.

In the track arrangement it may be seen that direct connection is afforded between each station track and each main track or the turntable, and that the most northerly station track serves as a switching lead for the freight tracks. The main switches and signals are controlled from the signal tower by a forty-one-lever electro-pneumatic interlocking machine furnished by the Union Switch & Signal Company, of Swissvale, Pa. The tower is of reinforced concrete, of ornamental design in keeping with the other work in the vicinity.

All passenger tracks are stone-ballasted, laid with the company's standard 91-lb. rail on creosoted ties, fastened with tie plates and screw spikes. For the other tracks 80-lb. rail is substituted and the stone ballast is eliminated, but the creosoted ties, tie plates and screw spikes are used. The turnouts and slip switches are in general No. 8's.

Pine and Bay Street Crossings

A full-width viaduct at Pine Street would have been very costly on account of the large number of tracks to be spanned, the close proximity of intersecting streets, making necessary numerous approaches, and the built-up condition of these streets. It was therefore arranged with the city to build at this crossing a footbridge only and divert vehicular traffic to the other crossings.

Bay Street is carried under the tracks, being bridged with a steel structure with sidewalk columns. The new grade of the street is lower than an old sanitary sewer in the street, to accommodate which one sidewalk was kept considerably higher than the roadway, the 20-in. sewer being rebuilt under the west sidewalk. The situation was further complicated by inability to settle the damage claims of the abutting property owners on the south side, and in order not to endanger the three buildings close to the street line in building the sewer the curb lines were moved out 5 ft. On the north side an additional story was built under each of the three dwellings nearest the crossing, the grades of the yards being lowered the necessary depth, and the grounds were otherwise improved. The roadways on both Bay and Grove Streets are paved with asphalt-bound macadam.

Included in the so-called Montclair improvement, to which the city contributed, was the elimination of the dangerous Bloomfield Avenue grade crossing. This crossing was not, however, directly involved in the terminal reconstruction, and will therefore be taken up in a separate article. The improvement cost in all about $500,000. In all of the work the city has co-operated with the railroad, handling all the damage adjustments and paying half of the expense involved, and has also contributed to other parts of the improvement, paying a total of about one-tenth of the entire cost of the work, exclusive of damages.

In carrying out the work one of the greatest difficulties was a common one in the reconstruction of terminals—the maintaining of traffic at all times. In addition to keeping the way open for the heavy morning and evening train movements, it was necessary to provide storage room over night for the cars and a means of heating them in the morning, and also to take care of the heavy milk and ice business, and with the limited total space available the construction operations were badly cramped.

Terminal Building, with Part View of Driveways, Park Spaces, Train Yard and Grove Street Viaduct

Personnel

The bridges, culverts and retaining walls were built by Reiter, Curtis & Hill, of Philadelphia, who also did the grading. Mr. A. E. Badgley, of Binghamton, N. Y., had the contract for the station. All other work, including track work, trainshed, platforms, station grounds, signal tower, Pine Street footbridge, boiler house, freight house, pavements, miscellaneous concrete work and temporary work, was done by company forces. The improvement was carried out under the general direction of Mr. G. J. Ray, chief engineer, and the more direct supervision of Mr. G. T. Hand, division engineer, assisted by Messrs. F. J. Nies, architect (who had charge of the design of the buildings and station grounds); A. E. Deal, bridge engineer; A. B. Cohen, concrete engineer, and W. H. Speirs, resident engineer on the work.

A Ship Canal between the rivers Wolga and Ob in Russia is contemplated by the Russian Government. The estimated cost is $86,000,000.

Pan and Raft Equipment for Water Evaporation Tests

The evaporating pan and raft shown in the drawings illustrating this article have been in successful use on various private and Governmental projects for some time past. As repeated requests for information about the outfit have been received by the designer, Mr. E. H. Kriegsman, of San Francisco, some of the details are here given.

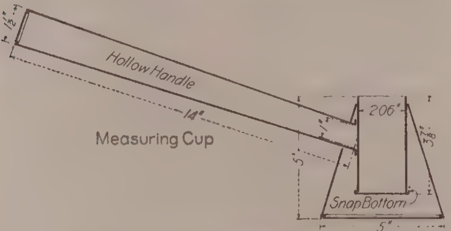
All timbers for the raft, except the decking, are 2 x 6-in. rough lumber secured by ½-in. wrought-iron bolts and corner brackets, no nails being used. The 1⅛-in. deck is bolted or lagged to 2 x 6-in. floor joists laid on 20-in. centers. At one end of the deck is a standard rain gage and a storage box with padlock.

Ordinary oil or paint barrels which are watertight must have the flotation determined before fastening in place, so that the 2 x 6-in. timbers are 3 in. out of water.

Pan

The pan is 3 ft. square, 18 in. deep and made of No. 18-gage galvanized iron, braced with 1 x 1⅛-in. angles. All joints are riveted and soldered watertight.

Two ¾-in. equalizing holes are placed near the bottom of the cylindrical still well in which a gage rod with a true needle point is stayed with a wire 3 in. from the top. As a protection the pan should receive a coat of dull-gray marine paint, followed by a second light finishing coat in thirty-six hours.

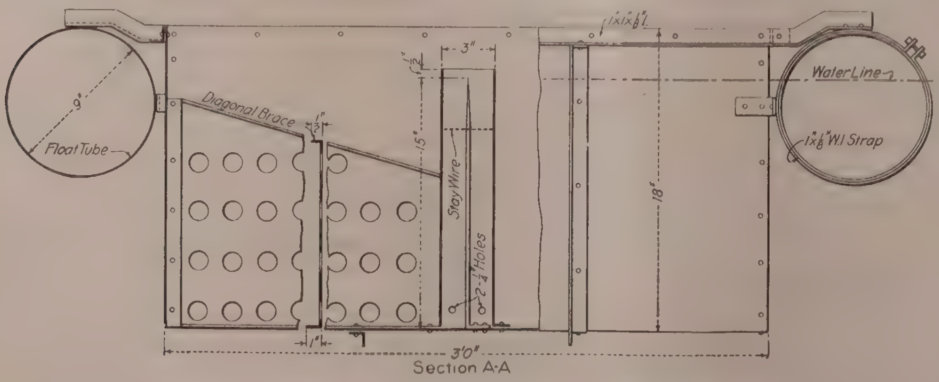


Double-Bottom Measuring Cup

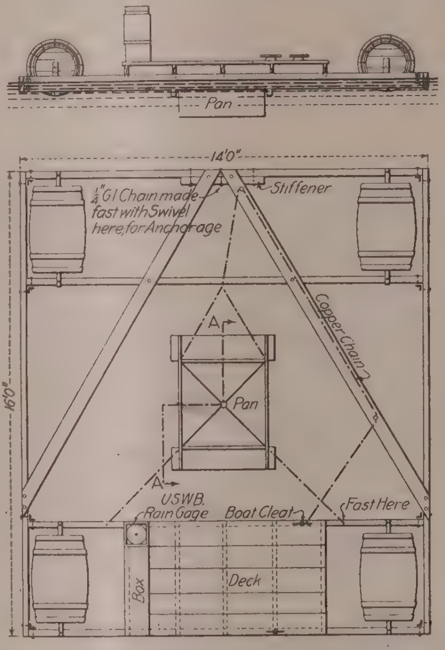
Measuring Cup

So that it will not sink when, by accident, it is dropped overboard, the measuring cup is made with a hollow handle and a double bottom of No. 28 or 30 sheet brass, with all joints soldered watertight. When filled with water the cup should weigh just 2700 grains more than when empty, a quantity corresponding to 12.96 cu. in., or a depth of 1/100 in. in the pan.

For an anchor a 250-lb. boulder may be drilled to receive a ring bolt. The ¼-in. galvanized-iron anchor chain should have a swivel near the bow fastening. Plenty of extra chain is desirable so that the anchor will not drag.



Details of Floating Galvanized-Iron Pan for Evaporation Tests



Plan and Elevation of Raft

Protecting Sandy Canal Banks

The first 50 miles of the main canal of the North Platte reclamation project are dug in soil of such light and sandy nature that it is easily drifted by winds or trampled down by stock, according to a report of the project engineer published in the "Reclamation Record." A portion of the canal extends through sand hills exceptionally susceptible to these agencies. It is not uncommon for an unprotected canal bank with a 12-ft. crown to be lowered 3 ft. within a period of six months, and the deterioration is rapid throughout most of the 50-mile stretch. In order to reduce the maintenance cost to a minimum the right-of-way was fenced against range stock and the canal banks were covered.

It was found that practically every method of canal-bank protection tried proved a failure if range cattle were allowed access. The cattle destroyed the vegetation, trampled down the banks and loosened the soil so that it readily blew away.

In 1909 a portion of the main canal in the sand hills was fenced. Prior to the building of this fence the adjoining sand drifts blew into the canal every winter and spring in large quantities. Since the fence has been built the sand hills within have become covered with sand grass and other protective vegetation, and the drifting of the sand into the canal has practically ceased, whereas the adjacent area on either side of the fence still consists of barren, glistening sand drifts. Since 1909 a total of 53 miles of four-strand barbed wire fence have been built, which, together with private fences, incloses all those stretches of the right-of-way where protection is essential. The cost of this fencing per mile is $125. Of this amount $75 represents material and $50 labor.

Although fencing is of prime necessity for effective protection, it is inadequate where the soils are so loose that vegetable growth cannot take hold. The fencing for protection has, therefore, been supplemented by the covering of the banks with manure or straw, gravel, brush and Russian thistles.

In October, 1909, 12,000 lin. ft. of newly prepared bank were covered with a light coat of stable manure at a total cost of $180, or about 1⅓ cents per linear foot. On March 1, 1910, this covering was in fair condition, but by August, 1910, it had nearly all disappeared through the action of wind and stock. The manure covering has to be renewed frequently, and even then the wind soon cuts holes through it and begins to undermine it and gradually to carry it off. Straw covering was tried with similar results, and it will not furnish even temporary protection until it is spaded or disked into the ground.

About 8½ miles of bank have been protected with a 3 to 4-in. layer of gravel on the top and outside slopes. This layer costs about 21 cents per linear foot, or 81 cents per cubic yard. This gravel layer has proved to be the most permanent protective coat and has been used where the haul was not more than 1½ miles.

Brush has been used to cover the outer slope in some places where gravel or other protective material could not be obtained at reasonable cost. About ½ mile of the bank has been covered in this way at a cost of about 20 cents per linear foot. This covering has been quite satisfactory both for holding the bank and for catching drifting sand. It is, however, liable to destruction from fires.

Russian thistles and tumble weeds have been used very successfully. In some cases these weeds have been placed on the banks and covered over with woven wire, and in other cases they have been weighted down by throwing earth on top of them. In the former cases a single or double width of hog wire has been stretched over the outside slope of the bank, the wire staked to the ground and the thistles filled in underneath it. This method has proved very satisfactory both in stopping erosion and in promoting vegetation, as well as in strengthening the banks by catching drifting sand. This method of protection can also be used on banks that are too narrow, too steep or too ragged to be protected in any other manner. Approximately 5 miles of banks have been covered by this method at a cost of 11 cents per linear foot, 4 cents of which is for wire and 7 cents for labor. As the average width of the bank is 9 ft., the cost per square yard is about 11 cents. The latter method can readily be used where the banks are in reasonably good condition. Where the slopes are flat, the weeds are generally plowed in by beginning at the bottom of the bank with the plow and working up the bank, filling the furrow each time with weeds, so that about one-half of the length of the weeds will be covered by the furrow and the remainder will project over the ground. On steep slopes and in small patches the weeds are usually placed and covered by means of shovels. The use of weed covering has been very satisfactory in holding the bank and getting a new growth of weeds started on it. Approximately 60,000 sq. yd. of bank surface have been protected in this manner at a cost of 6.1 cents per square yard.

Experiments were started on the project during October, 1912, for determining the value of cement coating, calcium chloride, crude oil, sugar syrup and coal tar for protecting the canal banks from erosion, but none of these methods has proved satisfactory.

Brush is comparatively scarce in the vicinity of the canal and therefore more expensive than the Russian thistles. Gravel protection is in many cases impracticable on account of the long haul and consequent heavy expense. Russian thistles and tumble weeds pile up in the canal during the fall and winter months and their use is found most advantageous and economical in the protection of the banks.

A Modern Bunk House in a Logging Camp

The camp of the Bloedel-Donovan Lumber Company near Alger, in Skagit County, Washington, has been built at a central point, from which a large part of the company's holdings, aggregating about one billion feet, can be logged, according to a paper read by Mr. J. J. Donovan at the recent logging congress in Spokane, from which the following has been taken. The camp is situated at the junction of the company's logging railroad and the Great Northern branch line which is operated by the company under lease. This location made it possible to establish supply and repair headquarters for the fifteen logging donkeys, four locomotives and sixty cars operated by the company, and incidentally made it feasible to supply steam and electricity to the camp buildings.

A two-story bunk house to accommodate 100 men was designed with the purpose of making it cleanly, comfortable and sanitary. On the main floor are eight rooms arranged for eight men each and two rooms for four men each, in all of which a hard-finish plaster is used. The upper floor is equipped simply with bunk accommodations for thirty men. The rooms accommodating eight men are 15 x 26 ft. in plan and have a light central partition with a 3½-ft. opening for a passageway. Each man thus has a corner of his own, with a shelf and locker. These rooms have two large windows hung on pulleys and a small table and electric light for the use of four men. Each bunk is made of steel, 3½ x 6½ ft. in size, and is provided with a plain mattress, each man furnishing his own bedding. The bunk house is heated with steam and has hot and cold water in the lavatory, where fourteen men may wash at one time. Modern toilets are installed and drain into a septic tank 500 ft. away. In front of both first and second floors 12 x 36-ft. porches were built, and are much used by the men in good weather.

The bunk house cost about $4,000 complete, with the exception of the steel bunks, which amounted to $675 more. To this total should be added a part of the cost of laying a wooden pipe line for water supply. This pipe was to supply the entire camp of about 250 men, and 1 mile of it was laid, partly of 4 and partly of 5-in. pipe, at a total cost of $1,500.

A Corrugated-Iron Storm Drain 5 ft. in diameter and 2400 ft. long has recently been installed in Salt Lake City, Utah. The same type of pipe in half sections, 12 to 18 in. in diameter, is in use for waterways at street intersections. Corrugated-iron pipe was first used in Salt Lake City two years ago.

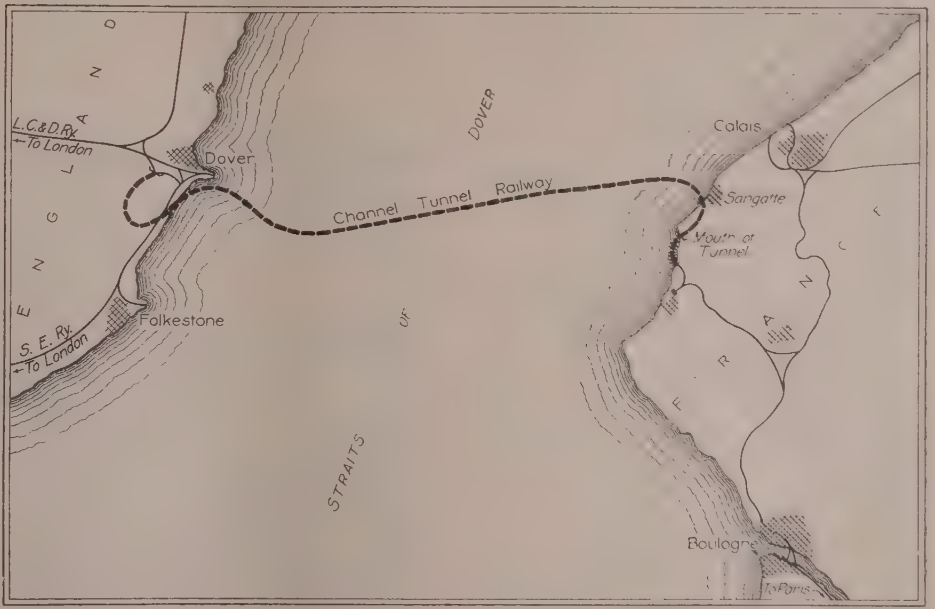
Proposed Tunnel under English Channel

Shield-Driven Twin Tubes, 24 Miles Long, and Approach Tunnel which Will Take Seven Years to Complete at an Estimated Cost of $77,000,000

By C. O. Burge, M. Inst. C. E., London

While America is completing at Panama the shipping connection between the two great oceans of the world financiers and engineers in Europe are seriously considering the project of connecting physically the two great nations of the Old World, and, incidentally, Great Britain with all Europe and Asia. In such great schemes the engineer must bow to the financier, and in the case of the Channel tunnel the vigorous advocacy of Baron Emile d'Erlanger—one of the leaders of the financial world in Europe and a man who appears to think in millions—is a strong guarantee that the $77,000,000 which are required for the scheme will be readily obtained.

Up to a few years ago the opposition to the tunnel came from the British naval and military experts, fearing invasion by that means, though even before that time the war authorities were by no means unanimous on this point. Lately the fear of successful invasion through the tunnel seems to have considerably modified, not only from the conviction of the difficulty of the operation itself, but from the fact that Great Britain cannot feed herself, and has only about six weeks' food supply in hand. An enemy, therefore, once getting command of the sea, would much prefer a blockade, and consequent threatened starvation, to bring to terms the previous "ruler of the waves" than sending the large army which would be necessary, with its impedimenta, through a constricted hole under the sea. Moreover, in the event of a blockade, the tunnel would afford, in the case of a friendly France, means of preventing, or at least lessening, the dread effects of national starvation by admitting supplies beneath the seas.



Proposed Location of Channel Tunnel to Connect England and France

A 24-Mile Tube

The present scheme for the tunnel, it is hoped, will come before the next session of the British Parliament, if it has time to spare from the consideration of much less useful legislation. There are to be two 18-ft. tubes in the subaqueous portion, 24 miles in length, constructed by the usual shield method, with cast-iron rings, backed by grouting and lined with concrete on the inside so as to present a smooth surface, and thus avoid obstruction in case of derailment. The approach tunnels will be of the ordinary land type. The alignment is shown in the drawing. After leaving the coast the tunnel is subject to two curves in order to bring it safely within the limits of the desired geological formation.

The descending grades of the British approach will be continued to about 6 miles from the coast line, which will be the lowest point. Thence there will be an easy rising incline to mid-channel, where the French half, similarly graded and constructed, will meet the British portion. The French company has already legal powers, and is, therefore, ready to begin work as soon as the British Act is obtained. Until the work is thus authorized the above and following details are necessarily subject to alteration, owing to further investigations which will then be justified.

Drainage Heading

Apart from the main tunnels, and of course preceding them in construction, will be an 11-ft. drainage heading, which will be at one side and at the same level as the main tunnels between the two lowest points previously mentioned. From there this heading will be on a descending grade to a shaft 350 ft. deep at each shore, from which the drainage will be pumped. It is not expected that this drainage will be excessive.

The geological system to which the strata of the locality belong is the Cretaceous. The tunnel will be driven through a compact chalk formation, most excellent material in which to work. This bed consists of white chalk, permeated with mud or clay, these two ingredients being very similar to the component parts of Portland cement. Fissures through the gray chalk may possibly be met with, though the geologists generally are against the probability of their occurrence. The engineers of the British half of the work, Sir

Douglas Fox & Partners, have had very great experience in the construction of tunnels encountering water pressure—among them the Simplon tunnel and the one under the River Mersey. They have carried out successfully grouting work under very high pressure, in order to deal with such contingencies. In fact, grouting under such high pressure as 500 lb. per square inch has actually been used in this connection. A case in point is that of the driving of the Astoria tunnel under the East River, New York, where pressures to this extent were used (see Engineering Record, July 26, 1913, page 95). In order to deal with water, pilot drills horizontally and vertically will be kept going in the headings as they proceed. As in the Simplon, the main tubes, which are to be 32 ft. apart on centers, will be connected at intervals with crossover headings provided with watertight doors for the purposes of construction, maintenance and ventilation.

The tunnels will be large enough to take the rolling stock of both countries, excluding the steam locomotives, which will be replaced by electric ones. At certain places, where the necessities of the work of construction or of the traffic demand it, an enlarged cross-section of the tunnel will be provided. Here the hauling machinery for removing the spoil and the pumps and ejectors for freeing the tunnel from water will be fixed. These will serve also as block stations for the signaling equipment when the traffic is running.

VENTILATION

It will be necessary to keep a sufficient supply of air throughout the entire length of the headings for the men employed. This will be effected by bratticing off the upper portion of the heading, thus forming a conduit of the required size for the volume of air, which would be blown in by high-speed fans. When the breakups are begun this conduit will also have to provide air for the men working at these enlarged faces until the second tunnel and crossways between the tubes are in progress. The ventilation will be effected in a manner similar to that adopted at the Simplon. In that case there are two parallel tunnels with connecting traverses. Fans blow the air in at the end of one tunnel, and after traveling up to the most recently excavated traverse, air returns by the other tunnel. This is known as the primary ventilation system; the volume of air is sufficient to keep all the galleries and traverses in a perfectly fresh condition. To ventilate the advanced end at the face, secondary ventilation will be adopted, so as to prevent effectually the stagnation of air so common in advance galleries of tunnels and mines, and enable the men to work with vigor and comfort.

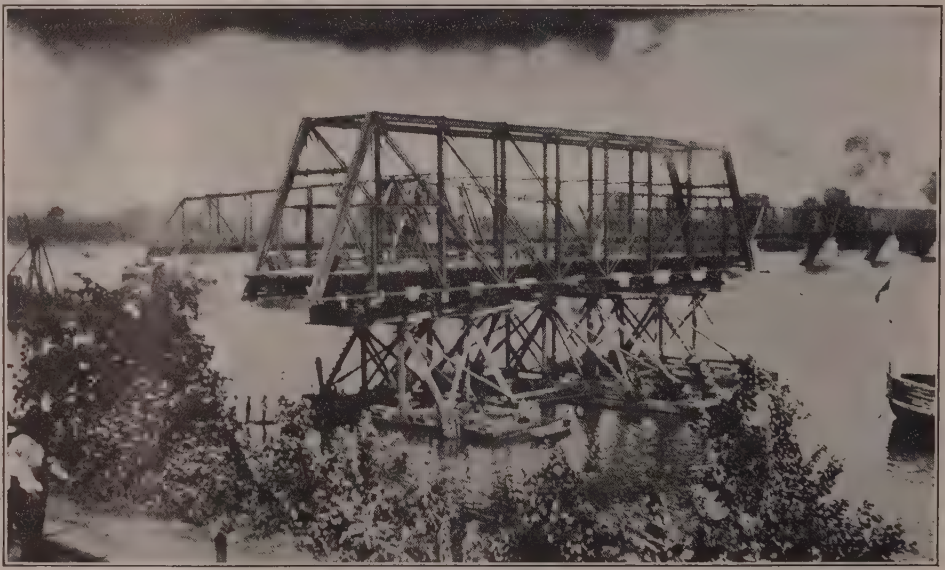
Turning to the permanent ventilation, means will be provided to furnish about 45,000 cu. ft. of air on each line of way, traveling in the direction of the traffic in each tube at a velocity of 6 ft. per second, which is a lesser provision than is required in many existing collieries. This will be sufficient to keep the air pure, even with a traffic of six trains per hour, carrying 500 people each. The drainage heading will be excavated by means of some approved cutter, or of Price's electrical digger, used in the tube railways of London. These machines work on a central shaft.

The tunnels will be lighted throughout by electricity and a separate and special circuit will be provided, so that in the event of the main traction current failing the lights in the tunnels will not be extinguished. Car lighting will be independent, each vehicle carrying its own store of light. Sanitary and hygienic regulations, similar to those successfully adopted at the Simplon tunnel, will be enforced during the progress of the works.

From local experiment and general experience it is calculated that the progress of the drainage heading will be 3 miles per year at each face, giving four years for its completion, and seven years is expected to be occupied in the total construction. No unusual construction difficulties are anticipated in the channel tunnel.

Shifting Railroad Bridge Spans on Barges

Another successful application of the method of floating bridge spans and arches was recently made during the reconstruction of the Wabash Railroad Bridge over the Mississippi River at Hannibal, Mo. The drawspan with two 180-ft. arms, weighing with its heavy oak highway floor about 550 tons, was moved transversely 180 ft. and seated on a new pivot pier. At the same time a 180-ft., 270-ton fixed span was moved into the location vacated by one arm of the drawspan. River and railroad traffic was maintained with as little interruption as possible and the change was successfully accomplished in two days.



Floating 180-Foot Railroad Span Shifted by Hoisting Engines

Four 24 x 70-ft. barges, heavily ballasted with water, were moored under the drawspan and timber towers, 33 ft. high, were erected on their decks. Owing to the falling level of water in the river it was necessary to supplement the towers by about 6 ft. of timber blocking to reach nearly to the bottoms of the vertical posts, four of which in each truss of each arm of the bridge were tightly wedged to bearing. Several of the diagonal tension members in the trusses were reinforced to take compression, and most of the water ballast was pumped out by four 6-in. siphons operated by means of steam from boilers installed on a pressure barge located near the pivot pier. The last few inches of water were removed by a 3-in. siphon operated by steam from the drawspan boiler.

Three 22 x 64-ft. barges and one 20 x 100-ft. barge, heavily water ballasted, were moored under the fixed span, and as the lower ends of the vertical posts were not adapted to take bearing on the blocking supported by the barge towers all of the floorbeams except the two end ones were wedged up on the blocking. The water ballast was removed by two centrifugal pumps and one pulsometer simultaneously with the pumping out of the drawspan barges.

Two barges, equipped with hoisting engines, were anchored at each side of the new pivot pier upstream from it. Manila lines were run from the hoisting engine on one barge to each of the barges under the fixed span and from the hoisting engine on the other barge to each of the barges under the drawspan.

At 11 a. m., Sept. 5, all of the barges had been pumped out and both spans were floating. The fixed span was moved first, being dropped downstream about 200 ft. by slacking off the manila lines, and was moored to two clusters of piles and to the west bank of the river. While it was being moved downstream a slight breeze drove it a little way upstream, but was finally brought back to position by lines leading to the west bank and to the steamboat, so that by 12 o'clock the span was safely moored.

Wire cables were run from the hoisting engine on the west anchor barge, led around the new pivot pier and attached to the upper end of the barges under the west arm of the drawspans. Similarly, wire cables from the hoisting engine on the east barge were led around the old pivot pier to the upper ends of the barges under the east arm of the draw, and at 3 p. m., Sept. 5, the drawspan was started and pulled to a final position about 5 o'clock. Some time elapsed before the old centers could be adjusted to the new centers on the new pivot pier.

The fixed span was left on the barges all night and the next morning the lines were rearranged, temporary blocking placed, and the fixed span moved to its final position. Although there was little clearance between it and the other span and a brisk wind was blowing, the movement was successfully accomplished and the first train crossed the bridge at about 3:30 p. m., a boat passing through the draw a little later.

On account of the low stage of the water the current was reduced to about 2 miles per hour and was more than offset by the brisk wind, which made it necessary to use power for pulling the spans downstream instead of upstream, as had been anticipated. The barges did not leak, and the spans appeared stable.

The work was executed by the Missouri Valley Bridge & Iron Company, of which Mr. E. H. Connor is chief engineer.

Design of Imhoff Sewage Plants—Part II

Suggestions Relating to Length and Cross-Section of Settling Compartment, Velocity of Flow, Detention Period and Scum Boards and Baffles

By Leslie C. Frank, Formerly of the Emscher River Board, Now Assistant Engineer, Baltimore Sewerage Commission, and Franz Fries, Engineer of Design and Construction, Emscher Board

The Imhoff tanks proper consist of settling chambers suspended over sludge-digestion chambers, the sludge entering the sludge-digestion chamber through slots in the bottom of the settling chamber.

There are three different designs of sedimentation chambers. The first is a longitudinal-horizontal-flow tank, and is the one most in use. The second is a downward-and-upward-flow tank. The third is a radial-horizontal flow tank. Neither of the last two designs has been adopted extensively as yet.

The downward-and-upward-flow tank was designed because it was thought that thereby a better cross-sectional distribution of flow might be secured, but the results of the operation of this tank on a large scale seem to indicate that its efficiency is not as good as that of the longitudinal-horizontal-flow tank. The reason for this may lie in the fact that the lower "slow-motion zone," mentioned elsewhere, is encroached upon by the deeply penetrating baffle, and that therefore the settling particles in the vicinity of the slot are not properly trapped.

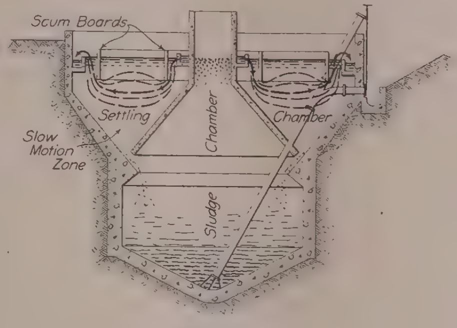
Radial-Flow Tank

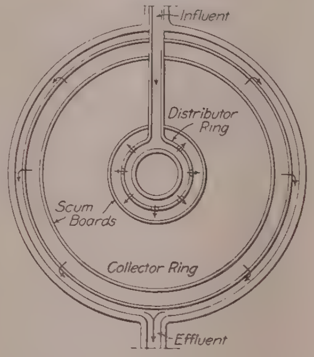
The first radial-horizontal-flow tank was constructed at Altenessen, and there are no operation data as yet. In this design the deeply penetrating baffle, causing downward and upward flow in the above-mentioned tank, has been removed and shallow scum-boards substituted therefor. A possible difficulty is that the radial direction of flow and consequent limitation of length of flow line may result in this flow line being too short to give the upper, rapidly moving currents sufficient opportunity to unload their sediment into the "slow-motion zone." It is obvious that the near surface velocities are not decreased in direct ratio to the decrease in length of tank, and hence these upper rapidly moving currents might have a detention period much too short for efficient sedimentation. However, the writers believe that if this radial-flow line is at least 14 or 15 ft. there will be no danger from this source.

A real advantage in the radial-horizontal-flow type is that here the velocity of flow, in order to provide the proper detention period, must necessarily be very low, and this fact will probably permit a greater amount of overloading than in the case of long-flow-line tanks, before critical velocities are reached.

As will be seen from the accompanying drawings, the settling chamber is inclosed by thin walls suspended in the main tank. The size of the settling chamber depends principally upon the necessary detention period and upon the volume of sewage. Generally, in the Emscher District the average daily volume of sewage is considered to flow off in fourteen hours. In America this is often taken at eighteen hours. The reason for this assumption is, of course, the relatively small night flow. From the thus determined hourly sewage flow and the number of hours' detention period the necessary volume of settling chamber is derived.

The detention period depends upon the character of the sewage to be treated. If the sewage contains a large quantity of suspended matters or coagulating chemicals, for example, a small detention period will suffice. Essen Nord has large quantities of iron wastes, and although the detention period is but one hour the settling effect is as good as in other plants whose sewage contains no such wastes, and whose detention period is as much as two hours. In general the detention period in the Emscher District is taken at one and one-half hours. Where the sewage is very dilute the detention period may have to be increased to as much as two hours. This is, in general, taken as the upper limit, as two hours will in nearly every case take out over 95 per cent of the solids capable of settling, and more than this may be dangerous from the standpoint of putrefaction of the sewage. Where the sewage flows to the plant irregularly the practice in Germany is to take a compromise detention period so as to avoid going too far above or below the maximum and minimum limits respectively. If possible, the sewage should be carefully examined with reference to the amount and character of its suspended content and with regard to the amount of character of trade wastes before deciding upon the necessary detention period.





Radial-Horizontal-Flow Tank

Length of Settling Chamber

It is also necessary to limit the length of the settling-chamber flow line, because if this is made too long the velocity necessary to give the proper detention period will be high enough to induce whirling currents and thus hinder sedimentation. Experience has shown that an average velocity of from 0.1 to 0.13 ft. per second is a reasonable maximum limit above which whirling is liable to occur, but in the design of Imhoff tanks it has been the practice not to exceed for dry-weather flow 0.03 ft. per second, so that the upper limit of 0.1 ft. is just reached when the storm-water dilution is threefold.

The maximum permissible length of flow line is limited solely by the required detention period and by the maximum permissible velocity. The flow line may be made shorter, within limits, than those factors determine because then the resulting velocity will be lower, and hence even safer, but it may never be made longer. Neither the older practice to design tank dimensions solely from the standpoint of velocity nor the present tendency in some cases to design them solely from the standpoint of detention period should be followed. Both of these should be considered. In the Emscher District tank lengths vary from 25 to 100 ft.

Cross-Section of Tank

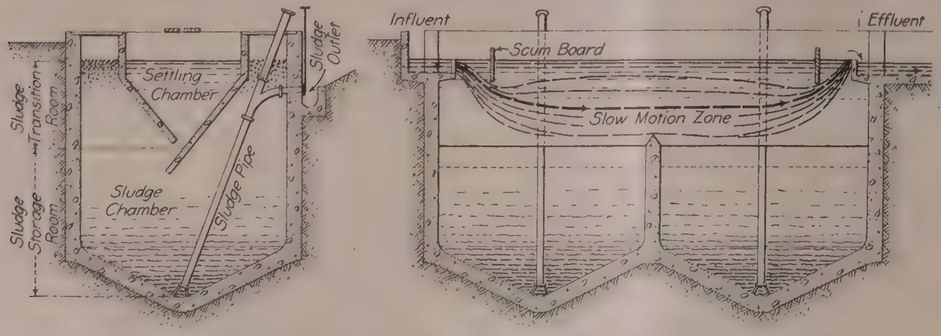
With the length determined the previously determined necessary tank volume will give the necessary tank cross-section. From the necessary cross-section the width and depth may be determined. In general, the width and depth of the tank should be governed by the necessity of proper distribution of flow. The movement of sewage in a settling tank has often been studied and described. One of the accompanying drawings shows approximately the direction, location and relative intensity of stream lines in Imhoff tanks. It will be clear that the velocity of flow decreases toward the lower part of a properly designed tank. The velocity in the vicinity of the slot must be sufficiently low so as not to stir up the sludge particles accumulated there, and should constitute the previously mentioned slow-motion zone. For this reason deep-reaching baffle boards are not advisable, and experience in this district has fully demonstrated this. The tank, therefore, should not be so shallow as to encroach upon this slow-motion zone and thus inhibit the passage of the settling particles through the slot. On the other hand, the tank should not be so deep as to inhibit proper vertical distribution of flow. In the former case the maximum possible percentage of settleable solids will not be removed, and in the latter case part of the volume of the tank will be ineffective.

The width of tank is also limited by the necessity of proper lateral distribution.

The following two examples show how the dimensions of Imhoff tanks as designed in the Emscher District are related to one another. In Holzwickede, at one of the smaller plants (one settling chamber over one sludge room), the length is 23 ft., the width 13 ft. and the depth 8.2 ft. In Essen Nord the length is 92 ft., the width is 28 ft. and the depth is 14.7 ft.

Inlets and Outlets

Since one of the most vital parts of settling-chamber design is the securing of a maximum efficiency of distribution over the chamber cross-section, the inlet and outlet ends of the chamber assume a corresponding importance. The current practice to reverse periodically the flow in Imhoff tanks of more than one sludge chamber, so as to obtain a uniform sludge in the various sludge chambers, makes it necessary in these cases that the channels serving the tanks be so constructed as to act alternately as influent and effluent channels. Hence their inverts must be at the same elevation. In addition, the weirs at the ends of the tanks must act alternately as inflow distributors and effluent weirs. Hence both of these weirs must be at the same elevation. Since the sewage flows over the effluent weir in a comparatively thin sheet the elevation of this weir practically determines the elevation of the sewage in the influent channel. This in turn determines with a given width of channel the fluid cross-section in the



Longitudinal-Horizontal-Flow Tank of Average European Proportions

channel, and hence, for a given dry-weather flow, the velocity in the influent channel. This velocity must be high enough to prevent sedimentation in the influent channel. Therefore the width of channel and the difference in elevation between the weir and the invert of the channel must be so adjusted as to secure this proper velocity, usually taken at about 2 ft. per second. At the same time, however, these two dimensions must also be adjusted so that during maximum flow (storm flow) the surface of the sewage in the same channel, when acting as the effluent channel, shall not be above the edge of the weir. This latter is to prevent variations in the tank water level. The water level of the tanks varies to a certain extent, of course, due to the varying heads on the overflow weir, but this variation, with sufficiently wide tanks, is small and negligible. If the outflow weir, however, is placed low enough to enable the water in the effluent channel to rise above it during maximum flows then the variations in the tank-water levels and the consequent exchange of settling-chamber and sludge-room content may be sufficient to produce disturbances in the digestion of the sludge.

Uniform Flow

It has often been attempted in the past to design the influent and effluent ends of a sedimentation tank in such a way as to secure uniform velocities of flow throughout the fluid cross-section of the tank. As has been previously explained, however, the existence of a slow-motion zone at the bottom of the sedimentation chamber is imperative in order that the settling sludge particles may be trapped. The velocity in this slow-motion zone should be considerably lower than the critical swirling velocity, and hence there is no reason why the velocities in the upper part of the tank may not be higher, within limits, than in the lower parts of the tank. It is simply necessary for the upper currents to have sufficient time to drop their load of suspended matter beyond the "pull" of the effluent device. Therefore the effluent device should logically be placed as high as possible —preferably a weir. It is evident that uniform cross-section velocities are illogical, and the influent and effluent devices should be designed accordingly.

The minimum width of slot is governed by the necessity of preventing choking, and the maximum width of slot is governed by the necessity of preventing undue diffusion surface. Generally, in the Emscher District, the width of the slot is made 6 in. to 1 ft., according to the size of the sedimentation chamber. The overlap of the slot should be at least 8 in., measured horizontally.

Scum Boards and Baffles

The penetration of scum boards under the water surface should not be so great as to render ineffective part of the upper volume of the chamber, or immoderately to increase the velocity in the lower slow-motion zone. In the Emscher District they are made to penetrate 12 to 16 in., and they usually project about 12 in. above the water surface. They are generally placed only at the inlet and outlet ends of the tanks, the one at the inlet end serving to collect the larger grease particles and to distribute the flow and the one at the outlet end serving to hold back the lighter floating particles. Intermediate scum boards are often inserted when the tanks are long. For example, tanks 100 ft. long would have one intermediate scum board. Intermediate scum boards are also of value in cases where more than ordinary quantities of grease come to the plant, as some of this is liable to slip under the first scum board.

Baffles are not considered advisable either in the upper or lower part of the sedimentation chamber. They have been tried, both from the standpoint of better flow distribution and in order to promote the production of the slow-motion zone. The conclusion has been that instead of this they decrease the cross-section of flow and tend to produce subcurrents, thereby hindering sedimentation rather than promoting it. It is true that some of the sewage flows through the tank in a much shorter period than the theoretical detention period. For example, in a tank designed for one hour's detention dye placed at the inlet end may be found to appear at the outlet end in fifteen minutes. It should be kept in mind, however, that this short detention part of the flow is practically always near-surface flow, which has generally delivered its load of sediment to the lower, more slowly moving part of the flow (slow-motion zone) before it reaches the outlet end. This is, in all probability, the reason why, despite the dye phenomenon, the performance of properly designed, non-baffled tanks is above 95 per cent of the settleable solids. A particular reason why deeply penetrating scum boards or baffles are detrimental to sedimentation is that they inevitably increase the velocity of flow in the lower part of the tank near the slot by locally reducing the cross-section, and thus tend to stir up and carry away such solids as may have accumlated on the sloping sides and are in the act of sliding down toward the slot.

One important point that is given special attention in the Emscher District is that the settling sludge particles are given absolutely no opportunity of remaining shelved upon projecting surfaces. All surfaces in the settling chamber (side walls, end walls, projecting pipes, etc.) are given an upper surface inclination of at least 1.2 vertical to 1 horizontal, and often 1.5 vertical to 1 horizontal.

[This is the second of a series of three articles on Imhoff plants by Mr. Frank and Mr. Fries. The third article, to appear in next week's issue, will discuss the sludge-digestion compartment and sludge disposal.]

Railroad Cattle Guards

In considering cattle guards the committee of the American Railway Bridge and Building Association assigned that subject went both into the necessity for them and the type of structure. Under the former head both the legal phase and the economic desirability were looked into. The questions submitted to the various railroads brought out a diversity of opinion. The committee deduced, however, the following conclusions, which were adopted by the association at the recent convention:

(1) Conformity to law in the installation and maintenance of stock guards is essential insofar as such conformity is a physical possibility without interposing any obstacles to the safety of train operation.

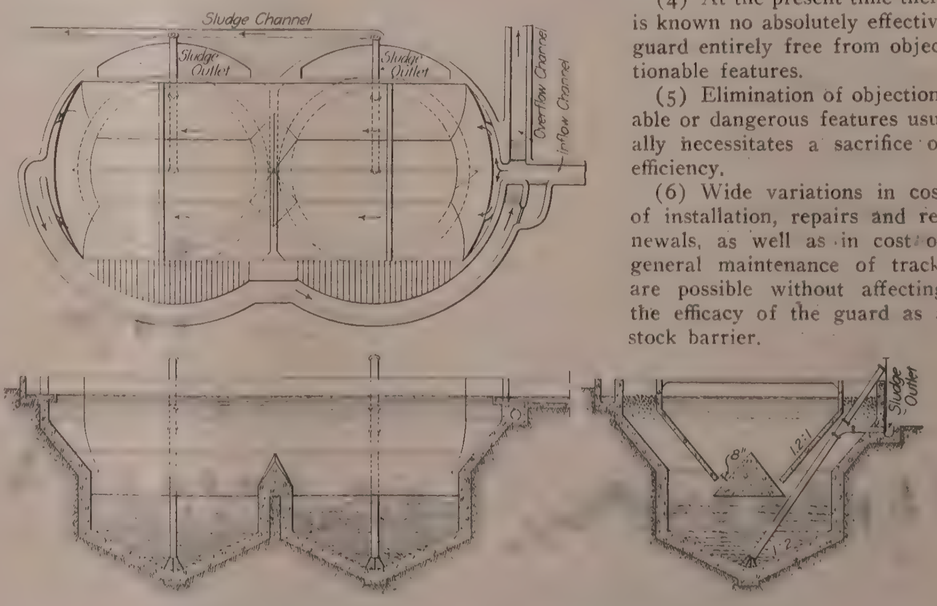
(2) Climatic conditions, statute requirements, range laws and scope of animal industry all have a bearing on the proper type of guard, and it would therefore seem impossible for any one design to meet satisfactorily the demands of the various localities.

(3) It is more difficult to erect positive barriers against stock accustomed to roaming wide areas of public domain than against more thoroughly domesticated animals.

(4) At the present time there is known no absolutely effective guard entirely free from objectionable features.

(5) Elimination of objectionable or dangerous features usually necessitates a sacrifice of efficiency.

(6) Wide variations in cost of installation, repairs and renewals, as well as in cost of general maintenance of track, are possible without affecting the efficacy of the guard as a stock barrier.



Longitudinal-Horizontal-Flow Tank of Average American Proportions

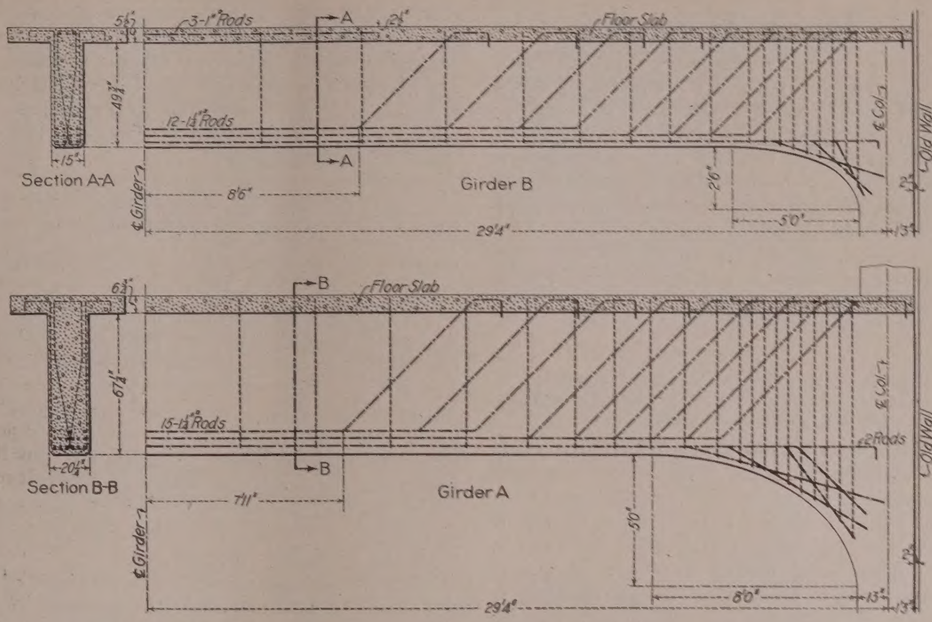
(7) Generally wooden-slat surface guards embody a minimum of objectionable features and are probably just as efficient as most types of metal surface guards.

(8) Aprons and at least the adjacent panels of wing fences should be maintained in white, and for this purpose paint is decidedly preferable to whitewash.

Reinforced-Concrete Girders of Unusual Span

Reinforced-concrete girders of 56 ft. 6-in. clear span are being used in connection with an additional garage now in course of erection to house motor delivery wagons for the Ward Bread Company on Pacific Street near Vanderbilt Avenue in Brooklyn, N. Y. The main portion of the addition to the existing garage is 61 ft. 2 in. x 130 ft. in plan, with a trapezoidal section wedged in between the rectangular portion and an oblique-angle corner of the old building. This extension will provide space for a stairway and an elevator, a doorway being cut through the wall to permit entrance by way of the old building.

It is the desire of the owners to maintain the full width of the addition entirely free of interior columns. To do this it was found impossible by any form of construction to bring the second floor of the new building on a level with the same floor of the old building and provide the clearance necessary for the gasoline trucks. Therefore it was decided to line up the second floor with the third floor of the old building, which is used as a repair shop. This arrangement gives a difference of elevation between the first and second floors of 24 ft. 2 in.; so economy in headroom was a matter of little importance in the design.



Reinforcement in Floor and Roof Girders of Concrete Garage

Columns and Footings

Ten columns support the eleven girders on the old building side of the addition; eleven columns are used along the opposite wall. The difference is due to an increased spacing of the two interior columns that adjoin the trapezoidal extension so as to accommodate the automobile elevator. All the columns, with the exception of these two, rest upon continuous, reinforced wall footings 2½ ft. wide by 3 ft. 10 in. deep. A ½-in. bearing plate was placed beneath each column. The two interior columns previously referred to are seated on isolated footings 30 in. deep by 7½ ft. square.

The second-floor girders are designed for a live load of 120 lb. per square foot. Their total depth, including the 6¾-in. floorslab, at the point of greatest bending moment is 74 in. They are 60 ft. 10 in. long over all and 20¼ in. wide. The column supports are each 26 in. wide. Fifteen 1¼-in. square bars with a mechanical bond in three courses furnish the tension reinforcement. Six bars spaced 3 in. on centers and 2⅝ in. from the face of the concrete constitute the lower reinforcement course. There are six bars similarly spaced in the middle course and three in the upper line. These latter nine bars are bent upward at an angle of 45 deg. at the exact points where the moment of inertia lines intersect the parabolic curve of bending-moment stress. A peculiarity of the construction is that the first four bars bent upward are not continued through to the column to reinforce the girder against shear but are bent over in an 8-in. hook. Of the bars in the lower course, two extend straight through and four are bent downward at various angles to produce a cantilever action in support of the girder. All bars were bent at the building by two ordinary rail benders. The maximum shearing stress at the points of support is taken care of by a bracket projection extending 8 ft. out from the face of the column. The vertical stirrups of ½ in. diameter are spaced from 6 in. to 3 ft.

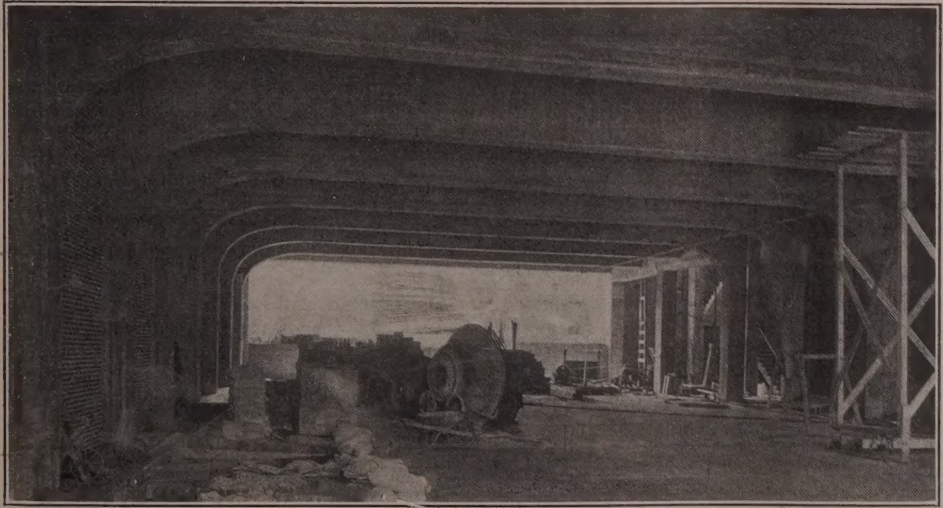
To provide an outlet for gasoline vapors that might collect between the deep girders and cause an explosion, 2-in. vents are carried up along one wall to an outlet above the roof.

Roof Design

The roof is a 5½-in. concrete slab overlaid by a 2-in. thickness of cork and a composition roofing. One end of every alternate girder is lower by 1 ft. 3 in. than the adjoining girder, and the drainage is carried off from the valleys thus formed through 4-in. rain leaders to the sewer. The roof girders are of slightly less depth, as they were designed for a uniform live load of 50 lb. per square foot. They are 55¼ in. deep and 15 in. wide. Compression reinforcement, consisting of three 1-in. bars 18 ft. long, is used in each girder.

All concrete was machine mixed, the mixer being located in the basement and charged from the ground floor. A bucket hoist carried the concrete to a bottom-dump hopper, from which it was wheeled as needed. The reinforced concrete was mixed 1:2:4 and the plain concrete in proportions of 1:2:5, Penn Allen cement being used throughout the construction.

The building was designed by Mr. C. B. Comstock, architect and engineer, of New York City, and is being erected by the Columbian Concrete Steel Bar Company, of New York, under the supervision of Mr. C. T. Lindsay.



Elimination of Interior Columns by 56-Foot 6-Inch Concrete Girders

Concrete Block Casting

A record in output has been established, it is claimed, by the traveling plant which is making the concrete blocks for facing the Kensico Dam of the Catskill Aqueduct at Valhalla, N. Y. The concreting tower is carried on railroad trucks, is equipped with three Ransome mixers, as described in the Engineering Record of May 10, page 528, and traverses three rows of Blaw-steel forms in which 4200 blocks per month are cast, equivalent to about 5000 yd. of concrete. The concrete is delivered from the mixers directly into the forms. No buckets or derricks are employed. A large number of blocks have already been manufactured and are piled up, many tiers high, adjacent to the casting yard to season before being placed in the structure. An important advantage of the plant is said to be the elimination of concrete buckets and the direct delivery of the concrete.

Book Reviews

A pamphlet entitled "Standard Specifications for Exterior Plastering" was recently published by the Associated Metal Lath Manufacturers. The booklet, which comprises 11 pages, deals with the materials, preparation of mortar, framing of the structure, mortar coats, finish, machine stucco and overcoating. It is written in specification form and will be of interest to architects and contractors. (Associated Metal Lath Manufacturers, Youngstown, Ohio.)

UNDERGROUND WATERS FOR COMMERCIAL PURPOSES. By Frank L. Rector. Cloth, 5 x 7½ in.; 98 pages; 8 illustrations. New York, John Wiley & Sons. $1 net.

From the title and the introduction it would be expected that the book would treat the commercial aspect of underground water on the basis of bottled water. As a matter of fact, however, little is said about this subject, the book being mainly a review of underground-water problems. Water analysis is treated in altogether too superficial a manner to be of use to the technical man, and yet too technical for the ordinary reader.

WATER PURIFICATION AND SEWAGE DISPOSAL. By Dr. J. Tillmans, Director of the Chemical Department of the Municipal Institute of Hygiene, Frankfort-on-Main. Cloth, 5¾ x 9 in.; 143 pages; 21 illustrations. D. Van Nostrand Company. $2 net.
(Reviewed by Weston E. Fuller, Consulting Engineer, New York)

This translation of a book by a German author is about equally divided between the subjects of water purification and sewage disposal. Water purification is taken up under two headings: (a) Purification on a large scale; (b) purification on a small scale. Under the first head the municipal problem is considered, while under the second head the problems of household purification and purification for manufacturing plants are dealt with. The chapters on filtration are, from the American standpoint, elementary. The impression of filtration gained from reading this book alone would be of a very crude process as compared with our modern plants. Illustrations of mechanical filters represent old types of small filters. The sand filters described are likewise very different from our modern works. Ozone treatment and other sterilization processes are more fully discussed, and are of interest as indicating German practice. Purification of water for iron and manganese removal is well treated, as is the treatment for carbon-dioxide removal. Purification of water for industrial purposes gives a good review of German practice in water softening, etc. The chapter on purification on a small scale for the household should prove of interest.

The sewage-disposal problem is taken up under three main headings: (a) Disposal of domestic sewage: (b) disposal of industrial sewage; (c) disinfection of sewage. Of the chapters devoted to domestic sewage, those which discuss screening, various types of settling tanks and the disposal of sludge are of most value. The chapters devoted to land treatment, filter beds, etc., contain less of interest to the American engineer. The chapters on the disposal of industrial wastes are exceptionally valuable, inasmuch as the author outlines briefly the character of sewage and the method of treatment of many different industries. These chapters, indicating as they do the European methods of treating trade wastes, should prove of value to American engineers and others interested in these important problems. The chapter on disinfection of sewage is very brief.

The book deals mainly with the principles of purification, little being given relating to design of works. Costs of various processes are given. The book is illustrated, well written, and of interest and value to the engineer who wishes to follow current practice in Germany in these subjects.

CONSERVATION OF WATER. By Walter McCulloh, C.E. Cloth, 7 x 10 in.; 99 pages; 40 illustrations. Connecticut, Yale University Press. $2 net.
(Reviewed by Weston E. Fuller, Consulting Engineer, New York.)

The volume contains the first of the Chester S. Lyman series of lectures delivered at the Sheffield Scientific School of Yale University. The first chapter discusses generally the problem of conservation of water for various purposes, showing what has been done along this line in the United States and what the possibilities are for future development. The limitation imposed by law, the effect of forests, the control of streams by the National Government and by States are discussed.

The second chapter describes the methods of obtaining the data necessary for the study of storage problems and flood control. The economic and geologic conditions, together with rainfall, run-off, evaporation and other hydrographic data, are discussed. The third chapter is devoted mainly to water power, giving statistics as to the amount developed and discussing the advantages of storage reservoirs in increasing the power developments. The fourth chapter takes up the advantages of storage for water supply, sanitation and irrigation purposes. It also discusses water waste.

Chapter five deals with the movement toward water conservation in New York State, outlining the work done by the Water Supply Commission. Many of the water-power developments of the State are described and the proposed methods of increasing the power of these developments and creating additional power at other sites are discussed.

The book is profusely illustrated with views of reservoir sites, power plants, diagrams and sectional drawings. In the main the volume is descriptive. A clear idea of the water-power situation in New York State is given and a general idea of the whole problem is conveyed. The book should be of interest to those concerned with the general problem of water conservation.

DER EISENBAU. Ein Hilfsbuch für den Brückenbauer und Eisenkonstrukteur von Luigi Vianello. Second edition rewritten and enlarged by Diplom-Ingenieur Carl Stumpf, Konstruktions Ingenieur an der Kgl. Technischen Hochschule zu Berlin. Cloth, 6 x 8½ in.; 687 pages; 526 illustrations. Munich and Berlin, Germany, R. Oldenbourg. 20 marks net.

The first edition of this book appeared in 1904. After the death of Mr. L. Vianello Carl Stumpf, professor of steel construction at the Technische Hochschule at Berlin, undertook the preparation of the second edition, which embodies the results of two years' work. Several chapters of an elementary nature have been eliminated. The book comprises one hundred and three chapters and is divided into eight main sections: 1, Elementary graphic statics; 2, statically determinate beams; 3, statically determinate trusses; 4, structures of three dimensions; 5, statically indeterminate systems; 6, masonry structures; 7, technical problems; and 8, practical problems.

The work is an up-to-date treatise on steel structures. The adaptability of influence lines and calculus and their superiority over older methods for calculating stresses are clearly shown in numerous examples. Especially for statically indeterminate and arch-shaped structures this feature is prominent. In Section 5 there is a treatise on portals, or panels without diagonals, where bending moments have to be provided for at the corner connections. Section 6 does not contain anything new and could well have been omitted.

The last two sections contain useful information for the detailer, but as is usual in German publications this part is not as efficiently treated as the theoretical part.

The work is logically arranged and the ground is well covered on theoretical subjects. It is apparent, however, that the author has written his book more as a reference work for the designer than for the student.

Letters to the Editor

The Engineer-Commissioner

Sir: Recently there has appeared in the public press notice to the effect that Hon. Charles A. Prouty, now a member of the Interstate Commerce Commission, is to retire at an early date to become director-general in an appointive position in charge of the national valuation work now begun. If this report is correct a vacancy on the commission board will occur. It is certain that one of the greatest problems ever placed before any body is now before this commission in the finding of the value of our railroad, telegraph and telephone properties. It further must be a fact that this is and will be the great question before this body. There is no member of the commission at present professionally equipped to treat with the matters at first hand, which in other words is saying that no present member can be said to be an engineer-economist of experience in such matters.

It is true that engineers and economists have been and will be employed extensively to carry out the great task, but the same can hardly be argued against the suggestion of the appointment of the new man who shall be an engineer competent and experienced in appraisal and economic matters. There are excellent reasons for offering this suggestion, and it is trusted it will be looked upon with favor by those empowered to name the new member. It is trusted also, if this suggestion meets with favor, that engineers generally will become active in its behalf. Our boards of health are comprised of doctors and specialists, while similar conditions are prevalent everywhere. In this situation why cannot a member of the commission itself be an engineer qualified to deal with matters first hand?

E. C. HURD,
Engineer, Nebraska State
Railway Commission.
Lincoln.

How Not to Build a Retaining Wall

Sir: I notice in your issue of Oct. 11, page 420, an article by Mr. K. A. Albert attacking the article of mine on "How Not to Build a Retaining Wall," which appeared in your issue of Sept. 20, page 324. His first criticism was answered in the short note I sent you before I knew he intended to make the criticisms which he did and which was published beside his artcle.

A detailed discussion of the various theories of earth pressure and retaining walls would take an amount of space which your paper would not be justified in devoting to the question and would perhaps lead to no very definite conclusion. It is almost needless to say that there are very divergent ideas on the subject among thinking engineers and there is room for a wide difference in honest opinion. I will content myself, therefore, with calling attention to certain inconsistencies in Mr. Albert's statements and will endeavor to meet him upon his own ground, without assuming any responsibility for the correctness of the computations which he made and published.

In the first place, he is curiously inconsistent in assuming an angle of repose of 32 deg. and then showing the fill back of the wall reposing at an angle of 45 deg. (a 1 to 1 slope). From his figure he has assumed an indefinite mass of earth at this slope and has computed the pressure of such a mass of earth upon a vertical plane extending 10 ft. 8 in. below the surface of the ground to be 1747 lb. The intensity of pressure is assumed to vary directly as the depth. To compare the two arrangements of walls it is only necessary to turn the wall around and locate it against this vertical plane, as shown by the heavy lines in the reproduction of his own figure, Fig. 1. The factor of stability, or factor of safety as he calls it, will equal 3.1 in this case as against the 1.46 which he computes for the original arrangement of wall.

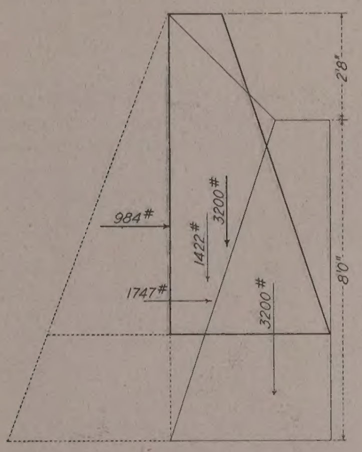


Fig. 1.

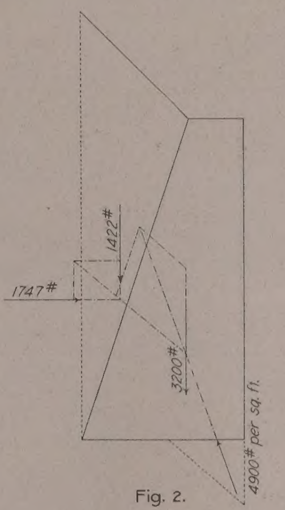


Fig. 2.

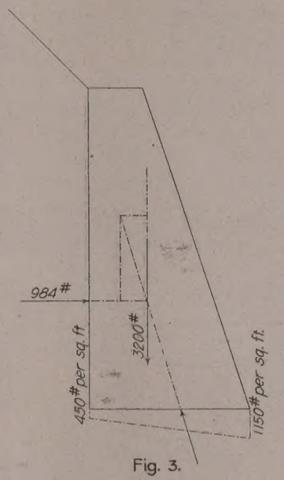


Fig. 3.

There is another factor which he has not mentioned, and that is the distribution of pressure over the bases of the walls. For the forces as he shows them the resultant acts about 7.5 in. from the toe of the wall and the distribution of pressure is about as shown in Fig. 2. It is, of course, impossible for any tension to exist between the earth and the bottom of the wall. The distribution of pressure with the wall turned in accordance with my suggestion is about as shown in Fig. 3. The resultant in this case acts at a point 3.5 in. from the center of the base. In such a wall as this the resultant should act as nearly as practicable through the center of the base. In this particular case this is very important, as the sea has washed much of the beach out along the old buttressed wall shown in the photograph accompanying my original article. The friction factor necessary to prevent sliding is 0.31 for Fig. 3 and 0.38 for Fig. 2.

Boston. Lewis E. Moore.

(The above letter was submitted to Mr. K. A. Albert for comment and his answer follows.—Editor.)

Sir: In the foregoing letter to the editor Prof. L. E. Moore answers a letter of mine, published in your issue of Oct. 11, under the title "How Not to Build a Retaining Wall." In the second paragraph of his letter Professor Moore states "that there are very divergent ideas on the subject among thinking engineers and there is room for a wide difference in honest opinion." A detailed discussion of the incorrectness of the existing theories, therefore, could be extended indefinitely.

In the following there is a description of a method used by the writer and it is of interest to know that the 35 to 60-ft. high retaining wall of the New York Connecting Railway recently built is designed under assumptions along similar lines but carried still further than those advanced by the writer. The work referred to was described in detail in the Engineering Record of July 9, 1913, page 80.

Professor Moore objects to the apparent inconsistency in assuming an angle of repose of 32 deg. for the calculation of the earth pressure and at the same time using 45 deg. for the slope of the surcharge. This, however, is quite correct, because almost any fill that does not contain too much water can be made to stand up on a slope of 1 on 1, and the vegetation that so quickly forms on such slopes maintains them and prevents hard rains from washing them down.

My assumption of any angle of repose of 32 deg. was simply chosen offhand, as the nature of the fill back of the wall in question is absolutely unknown to me. In the article of Sept. 20 Professor Moore indicates, by placing the earth pressure half way up on the back of the wall, that the slope of the surcharge is 1½ on 1, or an angle of 56 deg. 20 min. with the horizontal. If this actually is true I do not think that Professor Moore would think of using an angle as large as that for the computation of the pressures. It is better to assume too small an angle of repose than one too large in the computation so as to get maximum pressures, so far as the theory goes. When, then, the final pressures are determined theoretically it is the practice of the writer to lessen them 50 per cent, i.e., to use only half of the theoretically computed value. In doing this the specifications for the construction of the work must be drawn so as to prevent careless dumping of the fill back of the wall. This fill ought to be tamped or rolled in courses not exceeding 3 or 4 ft. so as to develop a certain amount of cohesion in the mass justifying the departure from the theoretical value of the pressure.

In my former letter I did not fully discuss the problem; I simply tried to prove that the wall was a stable structure and stated at the end that the resisting moment of the wall would be lessened 10 per cent if turned around. This is quite natural, as the wall in the latter position loses the increase in resisting moment due to the direct load of the fill on its inclined back.

In the last paragraph of his letter Professor Moore analyzes the pressures on the foundation and shows in a figure that the pressure at the toe of the wall is 4900 lb. per square foot. In using only one-half of the horizontal earth pressure, this pressure at the toe of the wall becomes about 2400 lb. per square foot, or 1.2 tons, which ought to be considered as very safe. The compression on the ground is then considered as distributed over 46.38 in. of the base, as the resultant force goes through a point 15.46 in. back from the face of the wall.

Professor Moore is right in his statement that the wall is not very well designed, but that it is a stable structure cannot be disputed.

New York City. K. A. Albert.

Flood Frequency and Flood Control

Sir: The editorial on "Flood Damage" in your issue of Oct. 11 indicates clearly that there are a number of important considerations commonly overlooked but which ought properly to be taken into account before large sums are invested in storage reservoirs for flood control. Such reservoirs are of undoubted utility for flood regulation, but whether or not they are the best remedy economically in a given case is a problem more complicated and difficult of solution than the average layman or perhaps engineer appreciates. It is generally impracticable actually to store the full volume of a large flood in the prisms of any system of reservoirs. The best that can be done, as a rule, is to utilize what may be called the spillway storage capacity, whereby the advance of the flood crest is retarded and the magnitude of the flood materially reduced at the reservoir itself. In some examples which the writer has studied it was found that 4 or 5 ft. storage above spillway level was of considerably more value than the entire volume of the prism of the reservoir below spillway level.

This probably is not always the case, but it suggests at once that surface area with regulated outflow is of much more value than depth or prism volume in a storage reservoir. The outflow over a spillway is proportional to the three-halves power of the depth on the spillway crest. The curve of discharge for a natural river channel can also generally be reduced to the form of an equation in which the flow varies about as the three-halves power of the depth of water in the channel.

The flood or overflow plain of a natural river channel corresponds very closely in its function and method of operation with spillway storage in an artificial reservoir. Natural storage on a flood plain has, however, this advantage, that it is usually obtainable in the lower reaches of the stream where the floods are greatest and the population most concentrated. Reservoirs, on the other hand, are usually available only in the region of the headwaters of a stream, and there may be sufficient inflow to the valley below the reservoirs to produce serious floods. These conditions are illustrated by the Mohawk River flood of March, 1913.

The maximum discharge at Cohoes near the mouth of the stream was 98,970 cu. ft. per second. The combined maximum discharge from the Mohawk River at Little Falls, from Schoharie Creek and East Canada Creek, draining a total of 2494 sq. miles, was 89,-

200 sec.-ft. The inflow to the valley from the remaining 978 sq. miles of drainage area above Cohoes was quite certainly at the rate of 50 sec.-ft. per square mile or more, which would make the total of simultaneous inflow to the valley 138,000 sec.-ft., or 40 per cent more than the maximum outflow rate. The storm producing this flood traveled eastward, so that the maximum discharge from the upper tributaries reached the mouth of the stream substantially at the same time as the maximum discharge from the lower tributaries. The large reduction in maximum discharge at the outlet as compared with the maximum inflow rate was due almost wholly to natural storage on the flood plain of the river between Schenectady and Little Falls.

The importance of this flood plain storage can be best appreciated by considering the volume of discharge of the flood for the five days March 25 to 29 inclusive. The aggregate discharge from 2494 sq. miles of the drainage basin during this period was 18,832,000,000 cu. ft. In addition the Delta reservoir on the upper Mohawk, having a capacity of 2,750,000,000 cu. ft., was partially filled. The combined capacity of the two large reservoirs in the upper Mohawk valley, the Delta reservoir, which is completed, and the Hinckley reservoir, which is under construction, is 6,197,000,000 cu. ft., or only one-fourth as much as the aggregate discharge from 70 per cent of the drainage basin of the Mohawk River for a period of five days.

Roughly estimated, the flood plain and stream channel storage in operation during this flood amounted to an average depth of at least 4 ft., and an average width of at least one-fifth mile for a length of 100 miles. This corresponds to a surface area of 20 sq. miles of storage, whereas the combined area of Hinckley and Delta reservoirs is 8.79 sq. miles. It is significant that much of this overflow land, especially in the upper Mohawk valley, is worth from $200 to $300 per acre and sometimes even more, for agricultural purposes, in spite of the fact that it is annually overflowed. The volume of flood plain and stream channel storage was probably not more than two or three billion cubic feet, or no more than the prism capacity of a good sized reservoir, but its regulative effect is very great, because it is distributed throughout the entire valley and receives directly the outflow from small tributaries subject to intense floods.

There is great lack of consistency in the practice of engineers in regard to provision for floods in engineering structures. On the same stream one finds some spillways or culverts adequate to take care of a flood that will not be exceeded once in a century, and others commonly built to take care of a flood which will recur at average intervals of 5 to 10 years. Following a suggestion of the late George W. Rafter, the writer undertook in 1896 to devise a rational method of analysis of flood records and other similar hydrological data. It is evident that a flood record covering, say a 10-year period for one stream, may contain a single flood which will not recur on an average once in 100 years; while another 10-year record may contain only floods liable to occur as often as once in 10 or 20 years. If, therefore, structures are designed on the basis of maximum observed floods in all cases, the results are bound to be inconsistent. The studies referred to resulted in the development of special probability curves or special flood formulæ and diagrams, containing time as a factor. This method was extensively used by the writer in determining high navigable stages for the New York State Barge Canal, especially in the western and middle divisions. These studies antedated by several years those of Mr. W. E. Fuller, referred to in your editorial. The method of investigation differed somewhat from that used by Mr. Fuller in that a special probability curve or flood formula was developed for each individual stream, showing the relation between magnitude and recurrence of floods on that stream.

These studies indicated that the analysis of flood records by special probability curves is in the line of making the best possibile use of all the available information and furnishes a method whereby all the structures on a given stream can be designed with reference to floods having the same average interval of recurrence. Studied by individual streams, the records of floods indicate some facts which probably would not appear in a similar study where the floods of several streams are grouped together. For example, a conclusion has been reached tentatively, contrary to the statement contained in your editorial, that there is no such thing as a maximum flood for a given stream; in other words, it does appear that there is a natural physical limitation to the flood-producing capabilities of each drainage basin. At the same time another and apparently somewhat contradictory fact appears. There seems to be a tendency both in the case of floods and rainfall for excessively large floods or rains to occur at more frequent intervals than the law of relation between magnitude and frequency of smaller floods on the same stream would indicate should be the case.

Albany, N. Y. Robert E. Horton.

100,000 Tons Dislodged by Blast

Sir: I am sending you two photographs that I think may be of interest. The blast picture shows a 10½-ton dynamite shot at our Martins Creek quarry. The face of this quarry is 170 ft. high. The dynamite was loaded in thirteen 6-in. holes that were drilled down the entire quarry face.

We estimate that the blast brought down from 75,000 to 100,000 tons of cement rock. The second picture shows the quarry after the blast. Formerly the rock here was worked down in benches, but since last summer we have been operating with heavy blasts.

S. Roland Hall,
Easton, Pa. Alpha Portland Cement Co.

Blast Dislodging 100,000 Tons of Rock and View of Quarry Face after Shot Had Been Fired